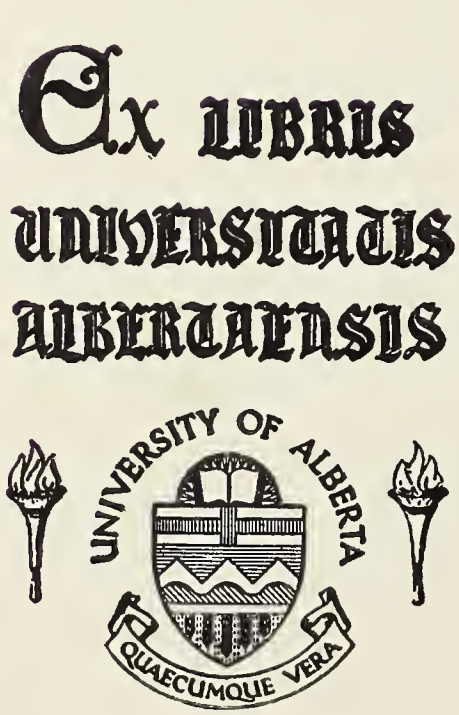
For Reference

NOT TO BE TAKEN FROM THIS ROOM

For Reference

NOT TO BE TAKEN FROM THIS ROOM



University of Alberta
Printing Department

Digitized by the Internet Archive
in 2018 with funding from
University of Alberta Libraries

https://archive.org/details/residualstrength00renn

Thesis
1966
R41

THE UNIVERSITY OF ALBERTA

RESIDUAL STRENGTH OF A CLAY-TILL APPLIED TO LITTLE SMOKY RIVER LANDSLIDE

by

ROBERT JOHN RENNIE

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE OF

MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

EDMONTON, ALBERTA

MAY 1966

ABSTRACT

The residual strength concept has been used to analyze slips in overconsolidated London Clay and it has been found that this strength closely represented that mobilized at failure. Many landslides in overconsolidated clays of Alberta cannot be successfully analyzed on a total or effective stress basis, but no attempts have been made previously to employ the residual strength. The residual strength was therefore obtained for a clay along the Little Smoky River and used to analyze the instability at a particular bridge site where serious movement has been prevalent.

The residual strength was determined by direct shear tests carried out to large strains. Triaxial undrained compression tests with pore pressure measurement were undertaken to compare with peak strengths from direct shear tests. Triaxial tests on remoulded specimens were conducted to investigate the similarity between remoulded and residual strength parameters.

A geological investigation of the bridge site revealed that the soil type tested was a clayey till and that the instability was in this formation. The residual angle of internal friction obtained from drained

direct shear tests was 22.6 degrees with zero cohesion. This angle agreed very well with the angle of internal friction representing normally consolidated strengths obtained from triaxial tests on undisturbed specimens. The remoulded angle was 26.6 degrees and was therefore not satisfactory in predicting residual strength.

Stability analyses conducted on two old slips gave factors of safety of 1.06 and 1.24 using observed ground water conditions. The safety factor of 1.24 became 1.04 if the water table coincided with the ground surface. There was good evidence that high pore pressures exist in the slope.

The Mohr circles obtained from triaxial compression tests indicated a preconsolidation load of 2.2 kg/cm^2. At higher test pressures, the residual and peak strengths closely agreed. Residual strength for this soil type did not therefore represent a dramatic reduction in strength as found for London Clay but nevertheless appeared to represent the in situ shearing resistance at failure.

Principal recommendations were that the effect of strain rate on effective strength parameters of overconsolidated clays should be investigated and that research should be continued into the shearing resistance of homionic, remoulded overconsolidated clays.

ACKNOWLEDGEMENTS

The sincere gratitude of the author is extended to Professor S. R. Sinclair for his continuing guidance and encouragement during the course of this research.

Many thanks are extended to the great number who provided assistance and in particular the author wishes to thank:

Mr. Glenn Smith of the Department of Highways of Alberta and the Research Council of Alberta for providing information such as site plans, air photos, movement data, borehole logs, etc.;

Dr. S. Thomson for helpful suggestions regarding the physical-chemical aspects of the problems;

Dr. J. F. Lerbekmo for interpretation and photography of thin sections;

my wife, Bernice, for typing drafts and for proof reading the final manuscript.

TABLE OF CONTENTS

		Page
Title Page		i
Approval Sheet		ii
Abstract		iii
Acknowledgements		v
Table of Contents		vi
List of Tables		ix
List of Figures		x
CHAPTER 1	INTRODUCTION	1
CHAPTER 2	LITERATURE REVIEW	4
	Strength Characteristics of Overconsolidated Clays	4
	Discussion	12
	Failure Criterion	13
	Pore Pressure and Volume Change	13
	Moisture Content, Strength and Consolidation Pressure	16
	Slope Stability	20
	Residual Strength	25
	Components of Shearing Resistance	31
	Summary	37
CHAPTER 3	GEOLOGY	39
	General	39
	Bedrock Formations	39
	Pleistocene History	42
	Character of Tills	50
	Summary	50

		Page
CHAPTER 4	APPARATUS AND PROCEDURES	52
	Aim of Test Program	52
	Direct Shear Box	52
	Instrumentation	54
	Calibration Direct Shear Box	58
	Procedure	61
	Triaxial Testing	65
	Summary	67
CHAPTER 5	RESULTS AND DISCUSSION	68
	Soil Properties	68
	Direct Shear Test Data	74
	Triaxial Test Data	84
	Moisture Migration	92
	Mohr Diagrams	99
	Residual Strength and Structure	109
	Moisture Content and Strength	114
	Summary	126
CHAPTER 6	STABILITY ANALYSIS	129
	Description of Site	129
	Movement	132
	Assumptions	133
	Discussion	134
	Stability Analyses	141
	Pore Pressure	144
	Summary	145
CHAPTER 7	CONCLUSIONS AND RECOMMENDATIONS	147
	Summary	147
	Geology	147
	Soil Properties	148
	Residual Strength	150

Page

Laboratory Procedures 152
Recommendations 154

List of References 157

Appendix A: Drawings of Shear Box and Trimmer A1

Appendix B: Sample of Printed Tape from Digital Recorder B1

Appendix C: Procedure for Direct Shear Tests. Procedure for Preparation of Thin Sections C1

Appendix D: Calibration Curve for Transducer D1

Appendix E: Borehole Logs E1

Appendix F: Typical e log P Curves F1
Sample Data Sheets for
a. Direct Shear Tests
b. Triaxial Compression Undisturbed
c. Triaxial Compression Remoulded

LIST OF TABLES

Table		Page
2.1	Factors of Safety Using Effective Stress Analyses	21
4.1	Characteristics of Direct Shear Box and Associated Instrumentation	60
5.1	Properties of Little Smoky Clay Till	70
5.2	Strength Data for Direct Shear Tests	75
5.3	Consolidation Data Direct Shear Tests	79
5.4	Triaxial Test Data	85
5.5	Measurement Errors Strength Tests	107
5.6	Variation in Angle of Internal Friction Due to Measurement Errors	108
6.1	Waterlevel Elevations in Boreholes and Piezometer Installations Little Smoky River	135

LIST OF FIGURES

Figure		Page
2.1	Krey-Tiedimann Modification of the Mohr-Coulomb Strength Equation	10
2.2	The Dependence of the Pore Pressure Parameter A on Stress History	15
2.3	Water Content Strength and Normal Stress Relationships for Clays	18
2.4	Comparison of Peak and Residual Strengths for Overconsolidated London Clay	19
2.5	Decrease in ϕ'_r With Increasing Clay Fraction	27
3.1	Stratigraphic Profile Along A.R.C. Cut Line (Figure 6.1) at Little Smoky River	44
3.2	Air Photo Illustrating Donut-Shaped Till Mounds at the Little Smoky Bridge Site	47
4.1	Sketch of Shear Box	53
5.1	Plasticity Chart for Little Smoky Clay Till 1962, 1965	71
5.2	Grain Size Distribution for Little Smoky Clay Till	72
5.3	Shear Stress and Vertical Deflection vs Horizontal Deflection in Direct Shear for Little Smoky Clay Till	80
5.4	Shear Stress and Vertical Deflection vs Horizontal Deflection in Direct Shear for Little Smoky Clay Till	81
5.5	Shear Stress and Vertical Deflection vs Horizontal Deflection in Direct Shear for Little Smoky Clay Till	82
5.6	Deviator Stress and $\bar{A}$ vs Per Cent Strain for Undisturbed Specimens	89
5.7	Deviator Stress and $\bar{A}$ vs Per Cent Strain for Remoulded Specimens	91

Figure		Page
5.8	Vertical Moisture Content Profiles For Direct Shear Tests on Little Smoky Clay Till	93
5.9	Moisture Content Profiles Along Failure Planes for Direct Shear Tests on Little Smoky Clay Till	95
5.10	Final Moisture Content Profiles for Triaxial Undrained Compression Tests on Undisturbed and Remoulded Specimens of Little Smoky Clay Till	98
5.11	Mohr Diagrams for Undisturbed and Remoulded Little Smoky Clay Till in Triaxial Compression	100
5.12	Comparison of Peak, Residual and Remoulded Strength Envelopes for Little Smoky Clay Till	104
5.13	Decrease in ϕ'_r With Increasing Clay Fraction	110
5.14	Thin Section of Direct Shear Specimen of Little Smoky Clay Till Rotated Under Plane Polarized Light	112
5.15	Thin Section of Direct Shear Specimen of Little Smoky Clay Till Rotated Under Plane Polarized Light	113
5.16	Moisture Content vs Log Strength for Little Smoky Clay Till	115
5.17	Moisture Content vs Log Pressure for One and Three Dimensional Consolidation of Little Smoky Clay Till	118
5.18	Modified Mohr Diagram for Triaxial Undrained Tests on Little Smoky Clay Till Undisturbed	119
5.19	Modified Mohr Diagram for Triaxial Undrained Tests on Little Smoky Clay Till Remoulded	120
5.20	Rendulic Diagram for Triaxial Undrained Compression Tests on Undisturbed Little Smoky Clay Till	122
6.1	Stereo Pair of Little Smoky Bridge Site	130
6.2	Cross-Section and Plan of West Bank Abutments, Little Smoky River	131

Figure		Page
6.3	Proposed Mode of Previous Slumping at the Little Smoky Bridge Site	139
6.4	Longitudinal Movement Pier 1 - 1965	140
6.5	Stability Analysis on Old and Existing Profiles at Little Smoky River Bridge Site	142

CHAPTER 1

INTRODUCTION

Inherent in the occurrence of a landslide is the fact that the stresses on the slip surface at failure should be equal to the strength of the soil through which the slide passed, or, the factor of safety should equal unity. Success has been achieved in obtaining factors of safety equal to unity for slips in normally consolidated and lightly overconsolidated clays, but overconsolidated clays have generally proven to be incapable of an accurate analysis. This has been true when shearing resistance has been expressed in terms of either total or effective stresses.

The cause of high strength that contributes to factors of safety in excess of unity has received much attention in the past, and two theories have evolved which have been successful. Hardy, Brooker and Curtis (1962) reduced the normal effective stress on the failure plane by the swelling pressure since overconsolidated clays of Alberta exhibit a large degree of rebound and swell in oedometer tests. Skempton (1964) successfully analyzed failures in slopes of overconsolidated London Clay using residual strengths. This latter concept has never been applied to landslides that have occurred in the overconsolidated clays

of Alberta. It was then necessary to reassess the instability at a particular site in Alberta where failure has occurred using residual strength.

The failure selected for the investigation involved serious movements under the abutment of a steel truss bridge situated on the Little Smoky River, approximately 35 miles north of Valleyview, Alberta. The bridge was constructed in 1957 and since that time, a pier and two abutments have been moving at a fairly constant rate toward the river. Recently, the movements increased severely and expensive corrective action was required to maintain the safety of the bridge. Stability analyses of the movement gave factors of safety greater than unity when shear strength was expressed in terms of both total and effective stresses. An analysis made using the swelling pressure to reduce the normal effective stress resulted in a factor of safety of 0.98 (Hardy, Brooker and Curtis, 1962).

The residual strength of the Little Smoky Clay was determined in a direct shear box in order to facilitate the achievement of the large strains after failure required to evaluate this strength. No tests were conducted previously using this apparatus, and in order to judge the validity of the results obtained, triaxial undrained tests with pore pressure measurement were also conducted to provide a basis for comparing peak strengths. Triaxial tests were also conducted on samples consolidated from a slurry to

investigate the suggested (Skempton, 1964) similarity between these strengths and the residual strength. The investigation was limited by lack of time for determining suitable strain rates for the triaxial and direct shear tests, additional field pore pressure observations at the site and a clear definition of the failure plane.

The investigation begins with a detailed review of shearing resistance and the problems regarding slope stability in overconsolidated clays. The geology of the site is then outlined with particular emphasis on the Pleistocene history. The apparatus is then described and procedures outlined. Test results are discussed in detail followed by a stability analysis using residual strength parameters. Lastly, conclusions are drawn and recommendations made.

CHAPTER 2

LITERATURE REVIEW

Strength Characteristics of Overconsolidated Clays

A brief review is made concerning the shear strength of cohesive soils prior to discussing slope stability. This review emphasizes the shear strength of overconsolidated clays as it is the laboratory shearing resistance of these clays that often exceeds the calculated shearing resistance at failure determined from slips in natural slopes. Overconsolidated clays are recognized as those having been consolidated under loads greater than those that presently exist in situ, whereas normally consolidated clays have never been subjected to a stress greater than that which exists at present.

The shearing resistance of cohesive soils has been expressed by the Mohr-Coulomb equation $\tau = c + \sigma \tan \phi$ where c is cohesion, ϕ is the angle of internal friction and σ represents the stress, normal to the failure plane. The Mohr envelope representing this equation is obtained by consolidated - undrained triaxial tests, but it has been found that strengths determined in this manner are usually higher than in situ, since reconsolidation in the laboratory results in a lower

void ratio (Bishop and Bjerrum, 1960). Besides, there is no requirements to find c and ϕ since undrained tests or, in particular, the unconfined compression test ($\phi = 0$) is more economical and has been found to closely represent the in situ strength of some clays. The undrained test gives the same strength for saturated specimens at the same moisture content, or in other words, ϕ is equal to zero. The $\phi = 0$ concept of strength has been found to closely reflect the short term strength of a clay or the strength immediately after construction. Nevertheless, research was conducted to determine strength parameters that were unique for a clay since the values of c and ϕ were susceptible to structural changes caused by sampling and the pore pressures developed in shear.

It was Terzaghi's hypothesis that shear strength and deformations of soils depended only on effective stresses which led to the concept of effective cohesion, c', and effective angle of internal friction, ϕ'. These strength parameters were considered unique for a particular cohesive soil. The Mohr-Coulomb equation was modified to $\tau' = c' + \sigma' \tan \phi'$ where $\sigma' = \sigma - \mu$ and represented the effective normal stress on the failure plane and μ was the pore pressure. There is much evidence, however, that c' and ϕ' may vary considerably, depending on the test procedures used to determine them. Some of the more common factors that affect the values of the effective shear strength parameters determined in the laboratory

are:

a. Strain Rate - Viscous Effect

It is well known that the undrained strength of cohesive soils decreases with decreasing strain rates (Taylor, 1948). The extreme effect of very low rates of stress application was studied by Casagrande and Wilson (1951) using undrained creep tests from which it was found that the undrained creep strength of heavily overconsolidated clay shales was approximately 40 to 80 per cent of the strengths obtained using normal strain rates. For specimens of Bear Paw Shale obtained from the South Saskatchewan River Project, the creep strength was 80 per cent of the peak value after 30 days.

b. Pore Pressure Measurement

If pore pressures are to be measured in triaxial undrained tests, a suitable strain rate must be selected in order to permit equalization of pore pressures in the specimen and to overcome the time lag in the pore pressure measuring system (Bishop and Henkel, 1962). The effect of the time lag in the measuring device can be readily taken into account and the problem has been greatly reduced at the University of Alberta by the use of transducers inserted at the base of the cells. This instrument requires a very low volume change to record pore pressure changes. The time required for the equalization of pore pressure can be computed using the consolidation characteristics of the specimen (Bishop and

Henkel, 1962). However, Bishop and Henkel have expressed doubt about the reliability of this calculation for over-consolidated clays that deform along narrow zones and there is evidence that strain rates should be much slower than estimated. Non-uniform deformations in triaxial specimens appear to be the principal cause of migration of pore water in undrained tests and non-uniform volume changes in drained tests. Even sands exhibit the latter characteristic and surprisingly, Shockley and Ahlvin (1960) found in triaxial tests that the volume increased in the failure zone of a loose sand specimen while decreasing elsewhere. The migration of pore water in cohesive soils has been discussed by Olsen (1960). Olsen concluded from the results of moisture content profiles taken vertically through triaxial test specimens, that for heavily overconsolidated clays pore pressures in the failure zone are negative and moisture, therefore, migrates away from the ends of the specimen during shear. For normally and lightly overconsolidated clays, the opposite trend in moisture migration occurs. The effect of not fully accounting for moisture migration in overconsolidated clays was described by Hirschfield (1960). He concluded that pore pressures measured at the base of the specimen would tend to be higher than in the failure zone with the result that the effective strength envelope would lie above that obtained, if full equalization had been allowed to occur. Probes may be used to overcome the problem of measuring pore pressures

at the base of the specimen, but the location of the probe may not be in the zone where the largest negative or positive pore pressure occurs. A trial and error procedure may be used to determine the correct strain rate, but this is complicated by the viscous decrease in the undrained strength mentioned in sub-paragraph a. Although there are many associated problems involved, it is essential that further research be conducted along these lines, since equalization of pore pressure appears to be a principal problem of heavily overconsolidated clays having low permeability, and it is these clays that are giving the high factors of safety in stability analyses.

c. Strain Rate - Drained Tests

If c' and ϕ' are to be determined by drained tests, the rate of strain must be sufficiently slow to allow for complete dissipation of excess pore pressures at failure. Hvorslev (1960) presented results which illustrated the effect of various strain rates on the shearing resistance obtained from drained tests. The shearing resistance of normally consolidated specimens increased with decreasing strain rates but decreased for overconsolidated specimens.

d. Partial Saturation

In undrained tests with pore pressure measurement, partially saturated soils may expel air bubbles that will block ports at which pore pressures are being monitored, giving erroneous evaluations of effective shearing resistance

(Bishop and Henkel, 1962). There is also the question concerning the value of the neutral pressure for a two phase air/water system in the voids of a partially saturated clay. Bishop, Alpin, Blight and Donald (1960) subtracted the pore air pressure from the total normal stress on the failure plane using a parameter x which varied from unity (S = 100%) to zero for a completely dry soil. The expression for the normal effective stress was $\bar{\sigma} = \sigma - \mu_a + x(\mu_a - \mu_w)$ where μ_a and μ_w were air and pore water pressure respectively. This method involved the separate measurement of the pore air pressure, which further complicated the laboratory determination of shearing resistance. Lowe and Johnson (1960) found that the undrained strength decreased with increased saturation and concluded that it was prudent to conduct tests under as high a degree of saturation as possible. Bishop, Alpan et al (1960) observed smaller values of c' and ϕ' in compacted soils that were back pressured to achieve 100 per cent saturation but it could not be definitely concluded whether strain rate, the use of high air entry porous stones or saturation caused this decrease.

e. Stress History

The strength parameters c' and ϕ' are dependent on stress history. The effect of preconsolidation load can be demonstrated by the Krey-Tiedemann modification of the Coulomb-Mohr strength equation which is $\tau = \sigma'_p \tan \phi'_c + \sigma'_f \tan \phi'_r$ where σ'_p is the preconsolidation load and

σ'_f is the normal effective stress on the failure plane (Hvorslev, 1960). The equation is illustrated in Figure 2.1.

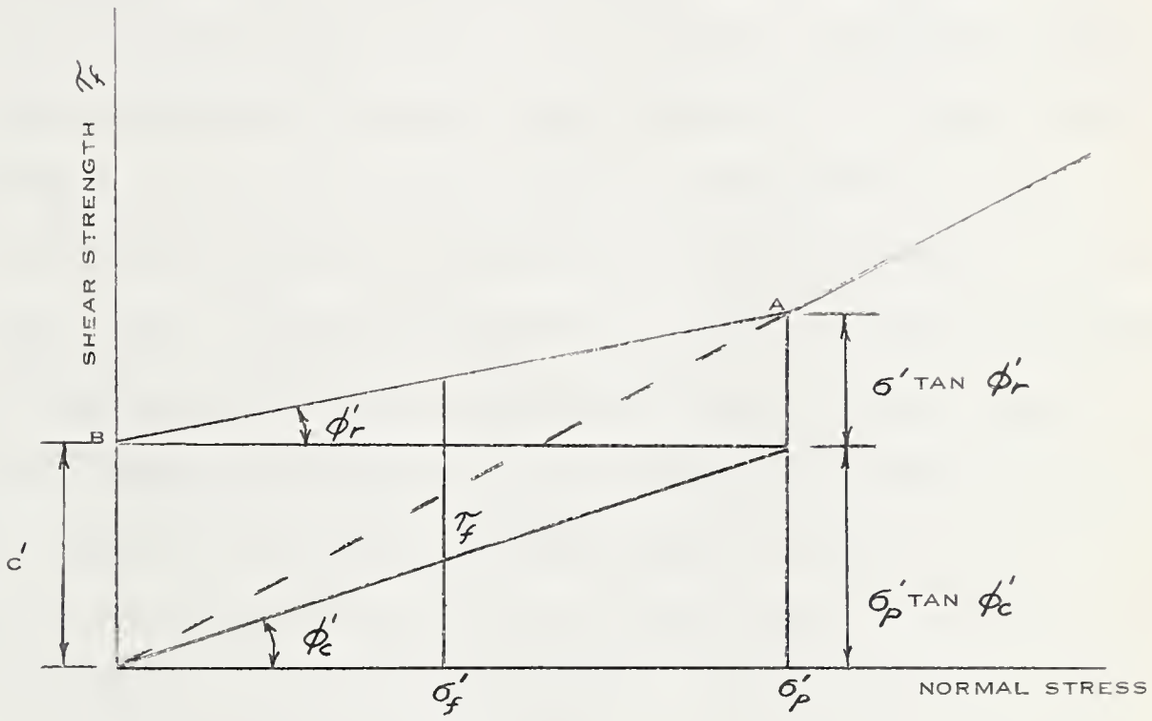


FIGURE 2.1 KREY-TIEDEMANN MODIFICATION OF THE MOHR-COULOMB STRENGTH EQUATION [AFTER HVORSLEV, 1960]

In the Krey-Tiedemann equation and in Figure 2.1 it can be seen that c' is dependent on σ'_p and that for other values of σ'_p, shear strength lines parallel to AB would be obtained. The straight line AB representing the overconsolidated strength envelope is somewhat empirical since test results have revealed a curvature of this envelope (Hvorslev, 1960).

f. Prestress Effect

The effective shear strength parameters c' and ϕ' can be determined in two ways; by consolidated-undrained triaxial tests with pore pressure measurement and by slow or drained tests. There is evidence that the parameters obtained from each of these tests may not be the same. Hirschfield (1960) found that the effective parameters determined from undrained triaxial tests tended to be higher than those from drained tests for normally consolidated clays that exhibited relatively high positive pore pressures. For such clays, the effective stresses in undrained tests decreased during shear causing a prestress that imparted additional strength. Bishop and Henkel (1962) indicated that this discrepancy could be overcome by comparing strengths after correcting for differences in the strain rates. However, overconsolidated clays showed drained strengths greater than undrained on an effective stress basis, especially if the specimen volume increased during shear. The drained test has extra strength due to the work done against the confining pressure, or normal load in the case of the direct shear test (Bishop and Bjerrum, 1960; Hvorslev, 1960; Wroth, 1958). According to Wroth, the surface energy or dilation correction may be difficult to apply to drained test results on heavily overconsolidated clays, since deformations in this case are usually confined to a very small zone. This infers that it is possible for a specimen to dilate

in a narrow zone in shear while other portions of the specimen are decreasing in volume. Shockley and Ahlvin (1960) found this to occur with sands. Moisture content changes in triaxial and direct shear specimens are investigated in this thesis to specifically determine if dilation is occurring and if it is reflected in volume changes or pore pressure measurements.

Discussion

Of the many factors that affect the determination of c' and ϕ' in the laboratory, the rate of strain appears to be the most important. At low rates of strain, the creep or viscous effect plays an increasingly important part. Strain rate appears to have a considerable effect on the measurement of pore pressure, particularly if deformations in the specimen are not uniform. A suitable strain rate is necessary to ensure that effective stresses are fully mobilized in drained tests. For moderate to heavily overconsolidated clays, it appears that the above mentioned factors will tend to decrease values of c' and ϕ' as the rate of strain decreases. For such clays, it is likely that progressively lower values of strength may be obtained until a strain rate representing creep is obtained for undrained tests. Drained tests on some clays also show a gradual decrease in strength for test durations exceeding that required for a fully drained test, an observation that is attributed to the viscous or creep

effect (Bishop and Henkel, 1962). It is then possible that the normal strain rates used in the laboratory to determine shear strength contribute toward the high shearing resistance obtained for overconsolidated clays and the author feels that investigations into the effect of strain rate on similar clays in Alberta would be worthwhile.

Failure Criterion

The failure criterion used in undrained triaxial tests may also have an effect on the strength parameters, c' and ϕ'. The failure criterion normally used is based on effective stress circles corresponding to the maximum deviator stress. Failure may also be taken at the point where the principal effective stress ratio $(\frac{\sigma'_1}{\sigma'_3})$ reaches a maximum. According to Bishop and Bjerrum, (1960), the difference in strength obtained from the two criteria is small for practical purposes. For overconsolidated clays that fail at large strains and exhibit a decrease in pore pressure during shear, the value of ϕ' obtained from stress circles representing maximum $(\frac{\sigma'_1}{\sigma'_3})$ will be slightly higher and this may be important when comparing drained and consolidated - undrained effective angles of shearing resistance.

Pore Pressure and Volume Change

Stability problems are of two classes with respect to pore pressure, namely, those problems in which the pore pressure is an independent variable and is controlled by

the ground water level or its flow pattern, and those pore pressures that are developed by the stresses causing instability. The stability of a natural slope is of primary concern in this research and with respect to the pore pressure developed in such slopes, Bishop and Bjerrum (1960) have stated, "Pore pressures are controlled by the prevailing ground water conditions which correspond to steady seepage subject to minor seasonal variations in ground water level. Natural slopes therefore fall into class (a) in which the pore pressure is an independent variable." It is therefore evident that once the parameters c' and ϕ' have been determined in the laboratory, the pore pressure, μ, measured in the field can be used to determine the effective normal stress on the failure plane by $\sigma' = \sigma - u$. In the case of highly plastic overconsolidated clays having a low coefficient of permeability, the measurement of the in situ pore pressure has presented problems. This was due to the long response time required for normal piezometric installations (Lindberg, 1965). Steps have been taken at the University of Alberta to reduce response time which has led to the development of a transducer piezometer, currently being field tested. Although it may be reasoned that the field measurement of pore pressures is the cause of not being able to obtain a factor of safety of unity for landslides in overconsolidated clays, experience has shown that the pore pressures required for a factor of safety of unity are unrealistically high (Hardy, Brooker and

Curtis, 1962). This indicates that the parameters c' and/or ϕ' as determined by conventional laboratory tests are not representative of the strength in the field.

Although test pore pressures are not representative of the hydrostatic and excess hydrostatic pore pressures in natural slopes, they are useful in indicating stress history and how the clay reacts to shear stresses. Skempton (1954) developed the relationship $\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)]$ where A and B are pore pressure parameters. The parameter $\bar{A} = AB$ represents the change in pore pressure with the change in deviator stress or $\Delta u/(\Delta\sigma_1 - \Delta\sigma_3)$. Values of $\bar{A}$ are very low for overconsolidated clays and may be negative, whereas values close to unity may be found for normally consolidated clays. This is illustrated in Figure 2.2 for a remoulded clay.

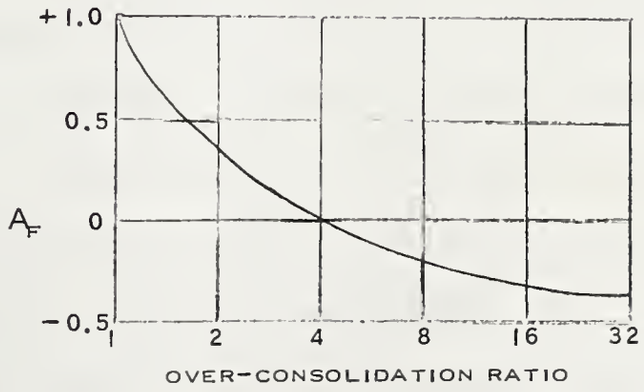


FIGURE 2.2 THE DEPENDENCE OF THE PORE PRESSURE PARAMETER A ON STRESS HISTORY [AFTER SKEMPTON, 1954]

The low values of $\bar{A}_f$ found in overconsolidated clays are the result of decreasing pore pressures at failure. Hvorslev (1960) states, "The pore water pressure decreases and/or the void ratio increases when strongly overconsolidated clays are subjected to an increase in shearing stresses." Normally consolidated and slightly overconsolidated clays exhibit increases in pore pressure with increase in shearing stresses. However, many of the expansive clays found in Alberta, that are considered to be heavily overconsolidated, exhibit increases in pore pressure prior to failure, but have low values of $\bar{A}_f$ (Hardy, Brooker and Curtis, 1962). It would appear that these clays do not dilate prior to failure and as mentioned previously, this apparent anomaly will be investigated in this research.

Moisture Content, Strength and Consolidation Pressure

Since attempts will be made to determine the migration of moisture within a specimen subject to shear by means of detailed moisture content profiles, it is worthwhile to investigate the moisture content-effective stress relationships of clays. This subject is treated in great detail by Hvorslev (1960) and only brief mention of pertinent points will be made here. Hvorslev has stated that strength is a function of both the void ratio and effective stress. This is evident from Figure 2.3 where strength and moisture content are plotted against normal effective stress. At

one normal effective stress, it is possible to have three strengths at each of three different moisture contents (degree of saturation = 100 per cent) depending on whether the clay is normally consolidated, overconsolidated (rebounding) or overconsolidated (reloading). Skempton (1964) has stated, "It is therefore possible to suggest, with perhaps a slight degree of over-simplification, that the residual strength of a clay, under any given effective pressure, is the same whether the clay has been normally or overconsolidated." In Figure 2.3, then, a strength test on an undisturbed overconsolidated clay would have a peak strength, say, at A and at further strain, a residual strength at B. From the consolidation-strength characteristics of cohesive soils, it seems reasonable to expect that the drop in strength from peak to residual must, therefore, be accompanied by an increase in moisture content at the failure plane from C to D, if no structural change occurs. If the Little Smoky Clay till to be tested is remoulded (normally consolidated from a slurry) its strength envelope should then coincide with the strength envelope OE that contains the residual strength, B. It should be noted that Skempton infers that the strength envelope for the normally consolidated specimens is based on ultimate and not peak strengths when speaking of the residual strength of normally consolidated clays.

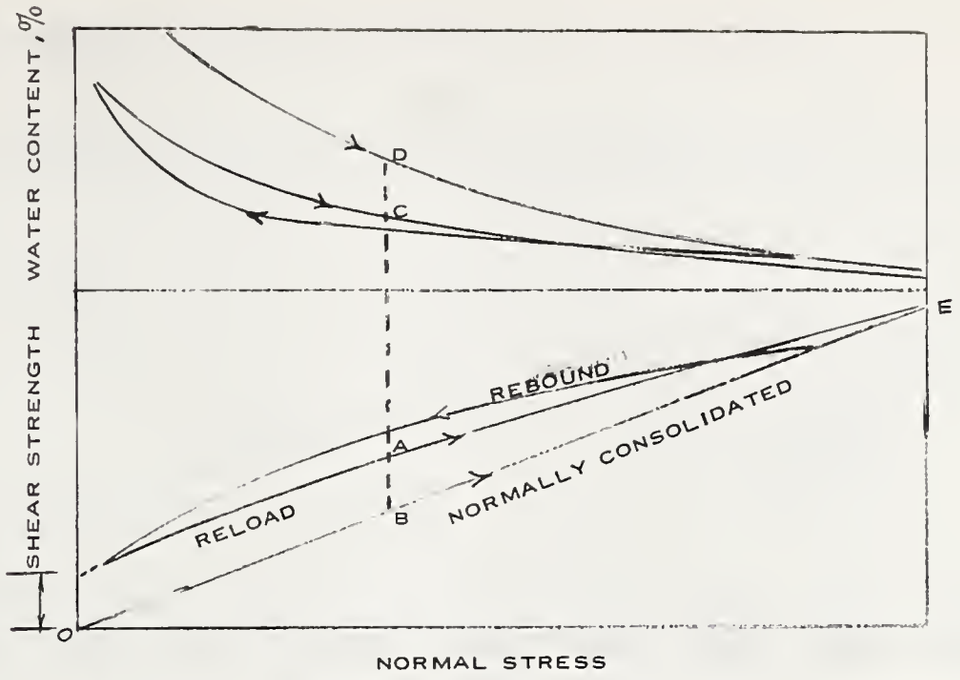


FIGURE 2.3 WATER CONTENT, STRENGTH AND NORMAL STRESS RELATIONSHIPS FOR CLAYS

It is well known that the curve formed by the relationship between moisture content and strength is parallel to the consolidation curve for a particular clay. It is difficult to visualize the strength curves for both the remoulded and undisturbed residual state being parallel to the normally consolidated consolidation curve in the upper part of Figure 2.3. The structure of the remoulded and undisturbed specimens likely would be very different at the same normal stress, especially when severe particle reorientation is associated with residual strength (Skempton, 1964).

A more complete picture of the relationship between the peak and residual strength Mohr envelopes is shown in Figure 2.4. These results were obtained from strength tests on overconsolidated London Clay in the low, medium and high

pressure ranges (Bishop, Webb and Lewin, 1965).

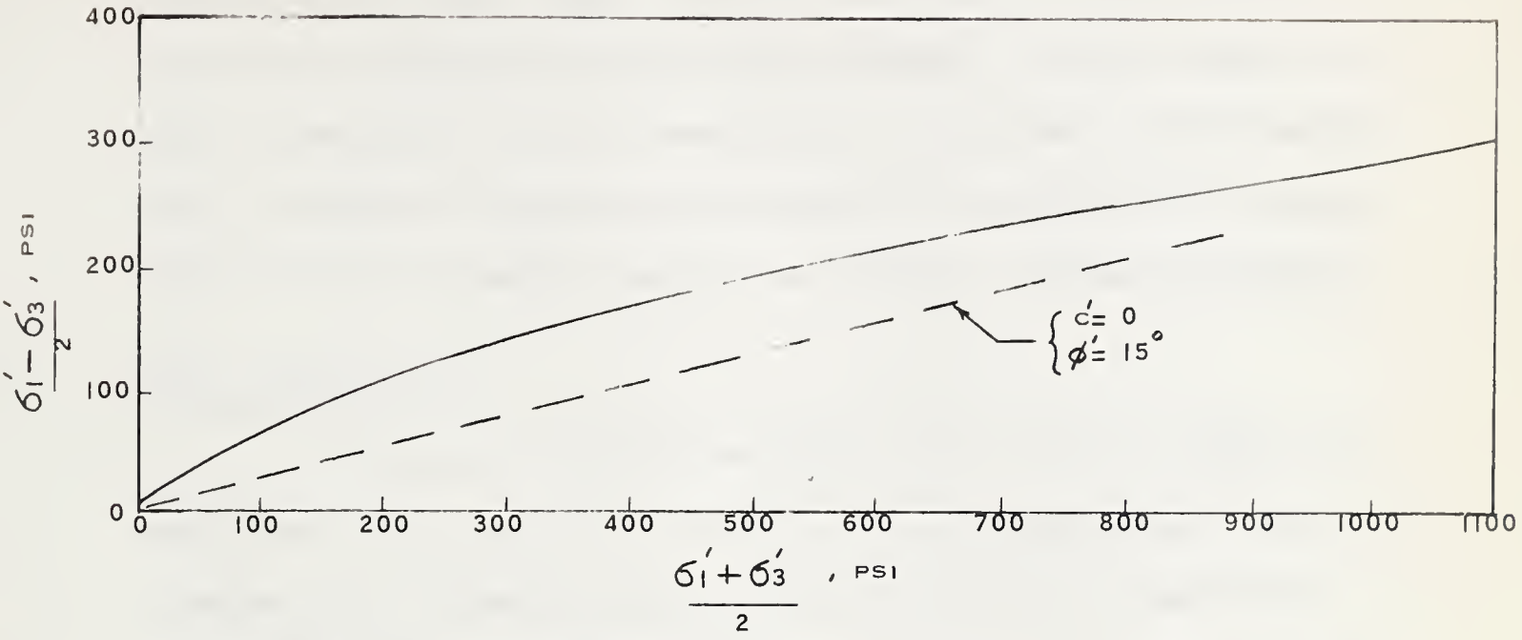


FIGURE 2.4 COMPARISON OF PEAK AND RESIDUAL STRENGTHS FOR OVERCONSOLIDATED LONDON CLAY [AFTER BISHOP, WEBB, AND LEWIN, 1965]

The strength envelope passing through the origin was obtained from tests on specimens normally consolidated from a slurry and some drained shear tests on undisturbed specimens carried out to large strains. These results tend to verify the correlation between normally consolidated strengths and the residual strengths of overconsolidated specimens.

Figure 2.4 also illustrates the fact that at effective stresses greater than the preconsolidation load, 900 - 1000 psi, residual and peak strengths approach the same value. In other words, the undisturbed clay is now acting normally consolidated at these higher stresses. It should also be noted that at effective stresses less than 100 psi,

the peak angle of internal friction for London Clay is considerably greater than the residual angle, and at zero effective stress, the clay has cohesion. Since tests in this research will be conducted at pressures less than 100 psi, a similar relationship should be obtained for Little Smoky Clay, assuming that it is heavily overconsolidated.

Slope Stability

It has already been mentioned that in stability analyses of natural clay slopes, the pore pressure must be treated as a separate parameter in conjunction with the effective stress parameters c' and ϕ'. This is due to the fact that the pore pressures in the slope have had an opportunity to come to equilibrium with the existing ground water conditions, a process that is time dependent, usually occurring over many years. Strengths in terms of total stresses ($\phi = 0$) are based on pore pressures that are stress dependent and are therefore restricted to the short term or "end of construction" problem. There is much evidence that total stress analyses of natural slopes in overconsolidated clays lead to a high factor of safety principally due to a marked decrease in pore pressure in undrained tests (Terzaghi, 1936; Peterson, Jaspar, Rivard and Iverson, 1960; Henkel and Skempton, 1954; Hardy, Brooker and Curtis, 1962). Since a long term stability problem in a natural slope is being considered in this thesis, further discussion

concentrates on effective stresses only and shearing resistance will be represented by $\tau = c' + \sigma' \tan \phi'$.

Stability analyses of natural slopes using effective strength parameters have shown some success in analyzing failures in natural slopes of intact normally and overconsolidated clays. Two case histories illustrating this fact are the Drammen Slide (Bjerrum and Kjaernsli, 1957) and Selset (Skempton and Brown, 1961). But effective stress analyses have, in general, resulted in too high a factor of safety for overconsolidated clays as indicated by the typical case histories in Table 2.1.

TABLE 2.1

FACTORS OF SAFETY USING EFFECTIVE STRESS ANALYSES

Soil	Effective Stresses*
1. Bear Paw Shale (Peterson et al, 1960)	3 - 4
2. Jackfield (Henkel and Skempton, 1954)	1.45
3. Selset (Skempton and Brown, 1961)	1.03
4. Dunvegan (Hardy et al, 1962)	2.81

* Fully mobilized c' and ϕ'

The slide at Selset was included in the table because it emphasizes what appears to be a predominant characteristic

of overconsolidated clays in England and that is fissuring. The factor of safety for Selset closely represented the conditions at failure and the authors suggested that the reason for this was that the "boulder clay" was comparatively intact. This was not the case at Jackfield where heavy fissuring was observed in the weathered zone, and the high factor of safety then inferred that fissures may be responsible for this poor assessment of failure using effective strength parameters.

The effective stress analysis for the Little Smoky landslide previously undertaken by Hardy et al (1962), yielded a factor of safety of 2.95 with $c' = 0.38$ kg/cm^2 and $\phi' = 19.5°$ and assuming that no pore pressures were acting. The pore pressure required for a factor of safety of unity was computed to be approximately 29 feet above the ground surface. Measured in situ pore pressures were much less than the required value. It appears reasonable, then, to assume that the effective stress parameters (c' and ϕ') did not represent the strength along the slip surface at failure.

Preliminary attempts to decrease the effective strength parameters were focussed on c' since it was found that cohesion would decrease with softening of the clay. The softening action that occurred in heavily overconsolidated clays was first described by Terzaghi (1936). Essentially, the softening was attributed to the seepage of runoff into cracks and fissures that were opened by lateral stress

releases. Absorption of moisture by the soil adjacent to the walls of the fissures caused uneven swelling and the development of more cracks, until eventually a slide developed in a natural slope. Skempton (1948) found that the softening occurring in overconsolidated clays took place over a considerable length of time. The undrained strength of London Clay decreased from approximately 2,000 lb/ft^2 to 500 lb/ft^2 in a period of 40 years.

The decrease in strength of natural slopes in London Clay was considered to be a long term phenomena. Eventually, all slopes in overconsolidated London Clay would be expected to attain the same angle of repose equivalent to a specific shearing resistance. To investigate this possibility, Skempton and Delory (1951) analyzed various natural slopes in London Clay and found no slopes steeper than 12 degrees, unstable slopes occurring between 10 and 12 degrees and all slopes flatter than 10 degrees were stable. In a landslide analysis at Jackfield, Henkel and Skempton (1954) found that a 10 1/2 degree slope was stable with $c' = 0$ and ϕ' unchanged. It was concluded then, that the long term shearing resistance of slopes in London Clay could be assessed with reasonable accuracy from laboratory strength parameters if c' was assumed to be zero.

The use of fully mobilized ϕ' and $c' = 0$ resulted in a factor of safety close to unity at Jackfield (Henkel and Skempton, 1954). Henkel (1957) investigated landslips

behind retaining walls and cuts and found that the factors of safety were unity for a value of c' intermediate between zero and that measured in the laboratory. He concluded that London Clay in this instance was not fully softened (c' = 0) when failure occurred. At Selset, the use of c' = 0 gave too low a factor of safety since suitable results were obtained with fully mobilized c' (Skempton and Brown, 1961). In the case of Little Smoky, the pore pressure required for a factor of safety of unity using c' = 0 was 20 feet above the slide surface which was 25 feet deep.

For London Clay, the concept of c' = 0 appeared to provide a suitable explanation of the mechanism of landslides. At the Little Smoky slide, however, the pore pressure required for a factor of safety of unity, assuming c' = 0, was never measured in the piezometer installations which were the Casagrande type embedded in sand layers. A similar situation in regard to pore pressure was also found in the analysis of the landslide at Dunvegan (Hardy, Brooker and Curtis, 1962). It was then necessary to determine some reasonable hypothesis that would result in a further reduction in the laboratory shearing resistance of overconsolidated clays.

In an attempt to realize the conditions at equilibrium at the Little Smoky and Dunvegan landslides, Hardy et al (1962) reduced the normal effective stress on the failure plane by the swelling pressure as measured in one-dimensional

consolidation tests. Void ratio-log pressure curves for clays from both of these sites exhibited substantial swelling pressures. Coulomb's equation representing shearing resistance in terms of effective stresses took the form of $\tau = c' + (\sigma - u - P_s) \tan \phi'$ where P_s was the swelling pressure. The factors of safety obtained for the Little Smoky and Dunvegan landslides, using fully mobilized ϕ' and c', observed pore pressures, and an average swelling pressure were .98 and .99 respectively and it was apparent that the reduction of the stress normal to the slip surface by the swelling pressure produced promising results.

Residual Strength

Skempton (1964) proposed that the shearing resistance of an overconsolidated, fissured clay slope would attain a reduced value, over a long time interval, equivalent to the ultimate or residual strength. This is the constant shearing resistance that most clays exhibit after failure in shear tests that are carried out to large strains.

In order to shear specimens to large strains, Skempton determined the residual strength of London Clay using drained tests in a direct shear box. The displaced portion was pushed forward and reversed several times until a constant value of shearing resistance was obtained. The peak strengths agreed well with drained triaxial test results, and could be defined by the Coulomb equation $\tau = c' + \sigma' \tan \phi'$.

The Mohr plot of residual strengths indicated a reduced angle of shearing resistance and little or no cohesion. In terms of the Coulomb equation, the residual strength was defined as $\tau_r' = \sigma' \tan \phi_r'$. Moisture contents taken at the failure plane after the test were higher than elsewhere and a thin, narrow strip adjacent to the failure planes showed severe particle reorientation. Considerable reorientation of clay particles and higher moisture contents were also observed in investigations of slip surfaces in natural slopes. The increased moisture contents were indicative of dilation and substantiated the reduction in strength previously discussed (Figure 2.3).

It should be made clear at this stage that the previously mentioned concept of c' = 0 is not the same as c' = 0 for residual strength. Recent evidence has shown that heavily overconsolidated London Clay exhibits a fairly sharp curvature of the peak strength Mohr envelope at normal pressures less than 100 psi (Bishop, Webb and Lewin, 1965). If strength tests were conducted at pressures less than this, the peak envelope, normally drawn as a straight line, would be steeper than the envelope representing residual strengths (Figure 2.4). The c' = 0 hypothesis is then associated wih a higher angle of internal friction than the residual angle. For London Clay, the difference between the peak and residual angle is about nine degrees (Skempton, 1964).

Skempton found that there was a definite relation-

ship between the residual angle of shearing resistance and the per cent clay fraction (<2 microns) for both normally and overconsolidated clays. The angle decreased with increasing clay fraction, as illustrated in Figure 2.5. Skempton proposed that the greater percentages of coarser equidimensional grains prevented, to some degree, the preferred orientation of clay particles along the failure plane which resulted in a higher residual strength.

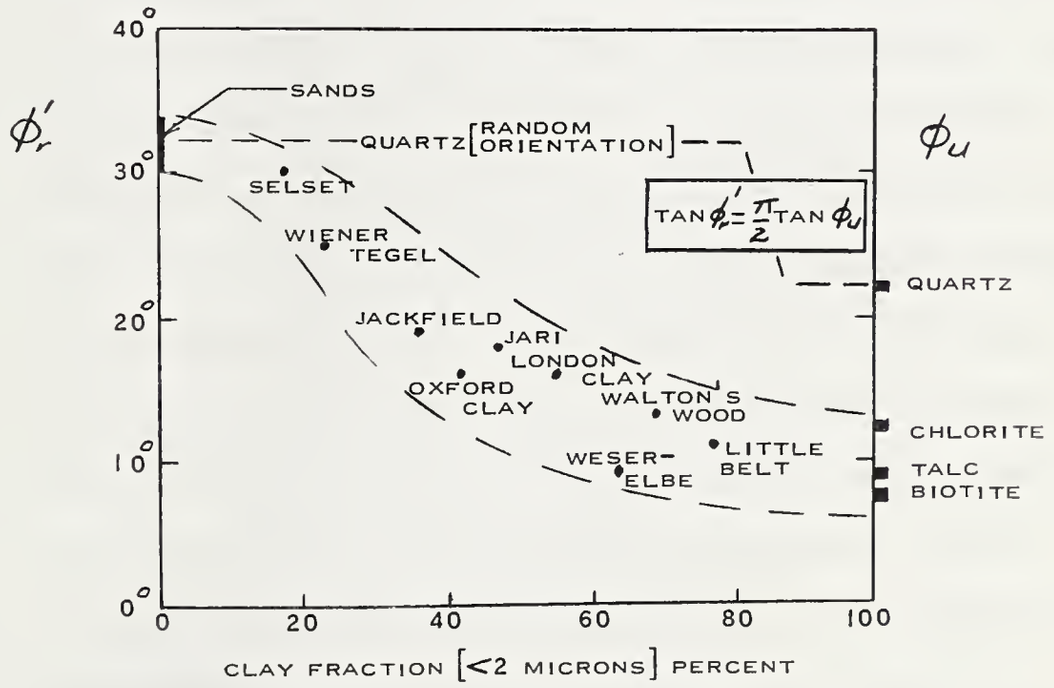


FIGURE 2.5 DECREASE IN ϕ'_r WITH INCREASING CLAY FRACTION [AFTER SKEMPTON, 1964]

Tests conducted on a sample remoulded and normally consolidated from the liquid limit gave a residual angle of shearing resistance equivalent to that obtained from undisturbed specimens. Skempton postulated that a specific clay had a definite residual angle of shearing resistance that was independent of its stress history.

It was necessary to describe the reduction of strength to a residual value in natural slopes. Skempton proposed the following causes of reduced strength:

a. Stress concentrations along fissures and cracks which caused local overstressing and a progressive decrease in strength. The stress concentrations caused local overstressing at points along a potential slip surface which then attained residual strength. Additional stresses were then shifted to other locations along the slip surface causing a further reduction in strength until the peak strengths were exceeded along the entire length of the slip surface. Failure would occur when peak strengths had been exceeded throughout and the majority of the slip surface was at the residual strength.

b. Shear creep which reduced the shearing resistance over the long term.

c. Seasonal fluctuations in moisture content and temperature. This was not considered to be too

serious a factor except that an increase in moisture content was indicative of an increase in pore pressure.

It is interesting that Skempton should mention creep as being partially responsible for the reduction in strength to a residual value. Although creep has been observed in natural slopes, it appears that emphasis has not been placed on it as being the single contributing cause of failure. The fact that creep reduces strength in overconsolidated clays has been observed in laboratory tests conducted by Casagrande and Wilson (1951) who found that the creep strength of Bear Paw Shale was approximately 80 per cent of the peak strength. It is interesting to speculate whether the creep strength is equivalent to the residual strength in this case. Ringheim (1964) reported that the peak strength for the soft Bear Paw Shale was $c' = 6$ psi and $\phi' = 20°$. The remoulded strength parameters were $c' = 2$ psi and $\phi' = 18°$. The remoulded parameters do not represent a reduction to 80 per cent of the peak strength, but the comparison may be faulty for two reasons: the method of remoulding was not specified and this may affect the strengths obtained; the reduction in strength due to creep is in terms of total stresses and it is speculative whether the reduction in the effective strength parameters, if any, would be similar or greater. No conclusion concerning this past point could be made by Casagrande and Wilson (1953). In this regard, it is interesting

to note that the final design parameters were c' = 0 and ϕ' = 9° determined from observed failures in natural slopes. If these parameters represent the residual strength, their prediction by testing remoulded specimens is considerably in error.

Skempton reassessed the slips at Jackfield and Selset using the residual strength concept. The results of analyses were expressed in terms of a "residual factor", R, which was equal to $\frac{\tau - \bar{\tau}}{\tau - \tau_r}$ where $\bar{\tau}$ represented the shear stresses acting on the slip surface at failure. From the expression for "residual factor" it could be seen that R = 1.0 if all stresses on the slip surface were equivalent to the residual strength at failure and R = 0 if stresses were equivalent to the peak strength. Using values of ϕ_r' = 19° and c' = 0 for Jackfield, R was equal to 1.12. The angle of shearing resistance representing stresses acting at failure was calculated to be 17° (c' = 0). The result indicated that the residual strength was approximately mobilized on the slip surface at failure. At Selset, R was computed to be 0.08. In other words, the peak strength represented by fully mobilized c' and ϕ' would have yielded a factor of safety equal to 1.0 whereas in terms of residual strength, the factor of safety would have been 0.69. As previously outlined, the good agreement between peak strengths and failure stresses for Selset was due to the fact that the clay was intact.

A retaining wall failure was presented as an example of how the shearing resistance of a natural clay slope decreased over the long term. The wall moved forward at a constant rate over a period of 29 years when the forward movement suddenly increased until failure occurred. The time rate of movement of the wall aptly illustrated the progressive weakening of the slope. A stability analysis resulted in a value of R = 0.61 or as concluded by Skempton, the slope lost 61 per cent of its shearing resistance in 29 years. Based on peak and residual strengths, the factors of safety were 1.6 and 0.6 respectively. A study of other case histories in fissured London Clay indicated that the angle of shearing resistance at failure varied from 15° to 18° with c' = 0. Skempton and Delory (1957) had previously determined that the stable slope angle of London Clay was 10° and Skempton found that this was equivalent to ϕ' = 16° and c' = 0 if the ground water table was at the surface. For London Clay, the residual angle of shearing resistance was equal to 16° (c' = 0) as determined in the laboratory. From this good agreement it can be seen that for London Clay, the stresses acting on a slope at failure were equal to the residual strength.

Components of Shearing Resistance

In many of the case histories mentioned and in the various approaches toward analyzing the stability of slopes

in overconsolidated clays, a change in one of the strength parameters has been assumed or has been observed in a laboratory test program. Cohesion was known to gradually decrease over the long term and this led to the $c' = 0$ analysis. The residual strength concept proposed a change in both the strength parameters in that c' approached zero and ϕ' was somewhat reduced from the peak value. Since the two components of shearing resistance appeared to be variable in the hypotheses outlined, a brief investigation is warranted into the physical-chemical reasons for the existence of cohesion and friction.

Cohesion has been generally described as being caused by the "stickiness" in the area of contact between adsorbed water hulls of adjacent clay particles. In recent years, studies of the electrochemical nature of clay particles has provided several causes of cohesion that are accepted by most workers in the field and these are (Lambe, 1960):

a. Van der Waal's forces of attraction,
b. Coulomb forces of electrostatic attraction and repulsion,
c. Hydrogen bonds,
d. Potassium bonds,
e. Cementation by carbonates, iron oxides, silicates, aluminates and certain organic matter.

These causes are basically forces that can be

mobilized between clay particles and they are found to be independent of the external forces acting on the particles. Usually the Van der Waal and Coulomb forces are considered to be the most important and these decrease rapidly in intensity or effect at greater distances from the clay particles. The Van der Waal's forces act over a much shorter distance than the Coulombic forces and the overall strength of both of these forces is dependent on the type of clay mineral, the soil structure, species of adsorbed cation, type and concentration of cations and, perhaps, anions, in the adsorbed and pore water, and temperature (Hvorslev, 1960).

The size and shape of the clay particle exerts considerable influence in determining the effect of the intrinsic or cohesive forces between clay particles. Basically, the degree of this influence is dependent on the combination of charge density and particle size. If the clay minerals, kaolinite and montmorillonite, are submerged in pure water, each will adsorb approximately the same thickness of water, but the ratio of the mass of adsorbed water to the mass of clay mineral is much larger for montmorillonite (Lambe, 1960). The ability of the montmorillonite clay mineral to retain large quantities of water in comparison to its size results in relatively large swelling and shrinkage characteristics which are usually detrimental to engineering works.

If an adsorbed cation is exchanged for one of higher valence there is a reduction in the forces of repulsion (Lambe, 1960). This concept has been substantiated by experimental work conducted by Rosenqvist (1955) on an illite clay. When considered on the basis of equal moisture content, Rosenqvist found that the strength of an illite clay was higher if calcium was adsorbed in place of sodium. Dahlman (1964) found that the strength of a montmorillonite clay increased in the order of adsorbed cation sodium, calcium, magnesium and potassium. This comparison was made on an effective stress basis. Rosenqvist (1955) also found that if the ionic balance between the adsorbed and free water was disturbed, there was a change in the interparticle forces. This was easily demonstrated by the classical experiment in which the concentration of free cations was increased in a clay-water suspension resulting in flocculation due to reduced repulsive forces. The effect of decreasing the concentration of cations in the pore water by leaching is just the opposite in that repulsive forces are increased giving a reduced strength (Rosenqvist, 1955).

The effect of structure on the intrinsic forces or cohesion appears to be the most applicable to the residual strength concept since experimental evidence indicates a rearrangement of particles in the vicinity of the failure plane. It has already been mentioned that Van der Waal and Coulomb forces are sensitive to distance and that Van der Waal's

forces are affected to a much greater degree. Computations have also indicated that a net increase in repulsive forces will occur if the distance between particles is decreased. Of more importance to the effect of reorientation of particles along the failure plane is the fact that the change from a perpendicular to a parallel arrangement decreases both the forces of attraction and repulsion (Lambe, 1960).

The factors that affect the friction component are dependent on the external forces acting on a clay specimen and friction is then considered to be mobilized by particle interference and dilation (Lambe, 1960). A portion of the particle interference is attributed to the actual physical interference between particles as is recognized in sands, but additional interference is caused by electrical variations in charge along the surface of clay particles. Accompanying this particle interference is dilation or an increase in void ratio as observed when a dense sand is subjected to shearing stress. Although this explanation of friction appears to separate effectively the physical-chemical causes that make up the two strength parameters, some of the causes may be interrelated. Lambe (1960) pointed out that two of the causes of cohesion, potassium and hydrogen bonding and cementation, have the effect of creating larger particles which could result in a greater degree of particle interference. Nevertheless, strength tests have been conducted using a procedure that allows for

the separation of the cohesion and friction components as defined by Mohr-Coulomb (Lambe, 1960; Dahlman, 1964). It was found that cohesion reached a maximum value at small strains and tended to dissipate as strain progressed. At higher strains, friction attained a maximum value. Lambe (1960) also found experimentally that compacted soils, which had a comparatively large value of cohesion, had a large stress-strain modulus.

Lambe (1960) developed an equation representing the effect that the forces of repulsion and attraction had in maintaining equilibrium in a clay specimen subjected to an external stress. The equation was $\sigma' = \sigma - u - (R - A)$ where R = electrical repulsion between clay particles, and A = electrical attractive forces between clay particles.

This equation infers that the R and A forces only affect the friction term of the Mohr-Coulomb equation, $\tau = c' + [\sigma - u - (R - A)] \tan \phi'$, yet it has been shown that they are primarily the cause of cohesion. In this respect, then, it is most difficult to apply the previously mentioned concepts to the observations that cohesion dissipates in a natural clay slope over the long term or that it disappears when direct shear tests are carried out to large strains. The most compatible mechanistic concept is the experimental observation that cohesion does tend to decrease at large strains, which agrees with Skempton's result in which $c' = 0$ at the residual strength. The best way to

eliminate many of the causes of cohesion is to increase the interparticle spacing which would particularly reduce both the R and A forces. This increase in spacing points to dilation in shear or water intake in a natural clay slope resulting from rebound during the geologic past. In the natural clay slope, the situation is further complicated by the cation exchange and changes in the cation concentration that may occur during rebound. For example, leaching could develop a specific type of structure in the clay and increase the repulsive forces as shown by Rosenqvist (1955). More research is required to produce a suitable correlation between physical-chemistry and the shear strength parameters c' and ϕ'. It is possible that cohesion and friction may not be different functions of strength and that a method other than the Mohr-Coulomb equation may be required to express shearing resistance (Dahlman, 1964).

Summary

This chapter was devoted to the strength characteristics of overconsolidated clays and their applicability to the evaluation of strengths mobilized at failure for landslides in natural slopes. Particular emphasis was placed on the range of parameters to be expected from overconsolidated clays subjected to shear stresses. Previous methods of explaining the apparent contradiction between shear strengths at failure occurring in the laboratory test and in natural

slopes were outlined. The concept of residual strength was discussed, and some case histories presented indicated that this concept had been successful in explaining the failure conditions in natural slopes of London Clay. An investigation of the pertinent rheological factors that affect shearing resistance was presented and an attempt was made to relate these factors to the residual strength hypothesis.

CHAPTER 3

GEOLOGY

General

Many of the river valleys in the Peace River district of Alberta were infilled with Pleistocene deposits overlying bedrock of Upper Cretaceous age. After an inspection was made of the samples from the Little Smoky bridge site, there was considerable doubt as to the formation from which the samples were obtained. This was primarily a problem in visual identification. The author had personally viewed the rock-like shales in the vicinity of Fort St. John, British Columbia and the shales at Dunvegan, Alberta, and both differed in appearance and texture with the Little Smoky soil. To assist in the identification of the samples, a review was then made of the pertinent literature concerning the geology of the Peace River area of Alberta. This review revealed the lithology of the samples taken and assisted in making conclusions about the instability at the site.

Bedrock Formations

In order of decreasing age, the bedrock formations of the Peace River area according to Russell (1943) are:

a. Lower Cretaceous

Fort St. John Group: A thinly bedded dark grey shale of marine deposition. The thickness of the formation has been estimated to be approximately 600 feet at the Smoky River near its conjunction with the Peace River. The formation thickens toward the West and outcrops can be seen in the valley of the Peace River in the western part of the Province of Alberta.

b. Upper Cretaceous

Dunvegan Formation: Consists of alternating sandstones and shales with beds that may exceed 50 feet in thickness. The formation is approximately 440 feet thick at the Smoky River. It is a brackish water deposit and only outcrops along river valleys west of Little Burnt River (18 miles east of Dunvegan Creek along the Peace River). The formation is exposed at the mouth of Dunvegan Creek at the Peace River and Rutherford estimated that it is covered by 10 feet of "boulder clay" at this location.

Smoky River Group: This formation consists of three members:

1. Kaskapau Formation: Thinly bedded dark grey to black shales similar to the St. John formation. This is the thickest member of the formation having a thickness of 850 feet.
2. Bad Heart Sandstone: This is a dark red sandstone that is coarse grained and is 15

to 25 feet in thickness.

3. Dark Marine Shale: Dark coloured, thinly bedded marine shale that is approximately 300 to 350 feet thick.

Wapiti Formation: Consists of sandstones and shales of fresh water deposition and is light grey to buff coloured. It averages between a few inches to 50 feet in thickness. This is the upper bedrock formation in the Peace River area and is poorly stratified which would make differentiation from a till difficult on this basis since the latter rarely exhibits stratification (Flint, 1963).

Allan and Rutherford (1934) have attempted to correlate other bedrock formations in the Province with the Peace River formation. Apparently Bear Paw Shale and the Edmonton formation which are Upper Cretaceous in age, are younger than the Wapiti formation but Bear Paw Shale is older than the Edmonton formation.

Rutherford (1930) has estimated that the bedrock strata dips to the south at a slope not exceeding 60 to 70 feet per mile. It has a strike of N 83° W. The topography south of the Peace River slopes downward in the direction of the Peace River. As a result, the older bedrock formations outcrop to the north and the younger ones to the south. An investigation of the bedrock outcrops along the Little Smoky River by Russell (1943) indicated that the Wapiti

formation was the uppermost bedrock formation at the bridge site on the Little Smoky River. Henderson (1958) attributed the slumping along major rivers and tributaries to the "soft" Kaskapau formation. This formation appeared to be too deep at the Little Smoky bridge site to have caused the bank instability. With 50 feet of Wapiti, 300 feet of Dark Marine Shale and 20 feet of Bad Heart Sandstone, the top of the Kaskanau formation would have been 60 feet below the existing average river level at the bridge site. This was assuming that the top of the Wapiti formation was at the same elevation as the top of the valley walls which was probably not the case, as Pleistocene deposition was extensive in the area.

Pleistocene History

The geological information concerning the Pleistocene and recent periods was obtained from Henderson (1958), except where otherwise noted. Henderson concentrated on Pleistocene and recent deposits in the Smoky and Little Smoky River basins south of the Peace River. Rutherford (1930) dealt with Pleistocene deposits in much less detail and concentrated on the bedrock formations.

The period of Pleistocene glaciation was preceded by a long period of erosion during the Tertiary period which resulted in the formation of wide valleys with gentle slopes

In early Pleistocene, the main rivers with sources in the foothills carried gravels deposited in them by Cordilleron glaciation. These gravels were in turn deposited over the bedrock river banks. It was estimated by Henderson that three and perhaps four ice invasions then occurred, originating from the northeast. Both erosion and deposition took place but the latter was predominant and valleys were infilled with sediments. After recession of the glaciers, valleys were again incised by rivers resulting eventually in the existing topography.

The preglacial Saskatchewan Sands and Gravels, as termed by Rutherford (1930), had their origin in the foothills of the Rocky Mountains. They contained neither granites nor gneisses of shield origin but commonly consisted of quartzites, cherts and sandstones. In early Pleistocene, these gravels were deposited over the bedrock river banks by fast flowing melt water from Cordilleron glaciers and, therefore, occur between bedrock and overlying till formations. According to Henderson (1958) they can be found in the Smoky, Little Smoky and Peace River valleys. They are also common in the North Saskatchewan River where they overlie the Edmonton Formation (Bayrock and Hughes, 1962).

A cross-section of the Little Smoky site is illustrated in Figure 3.1. Also shown are the soil types determined from the various boreholes. Gravelly clay deposits were found at the bottom of boreholes 1B and 5.

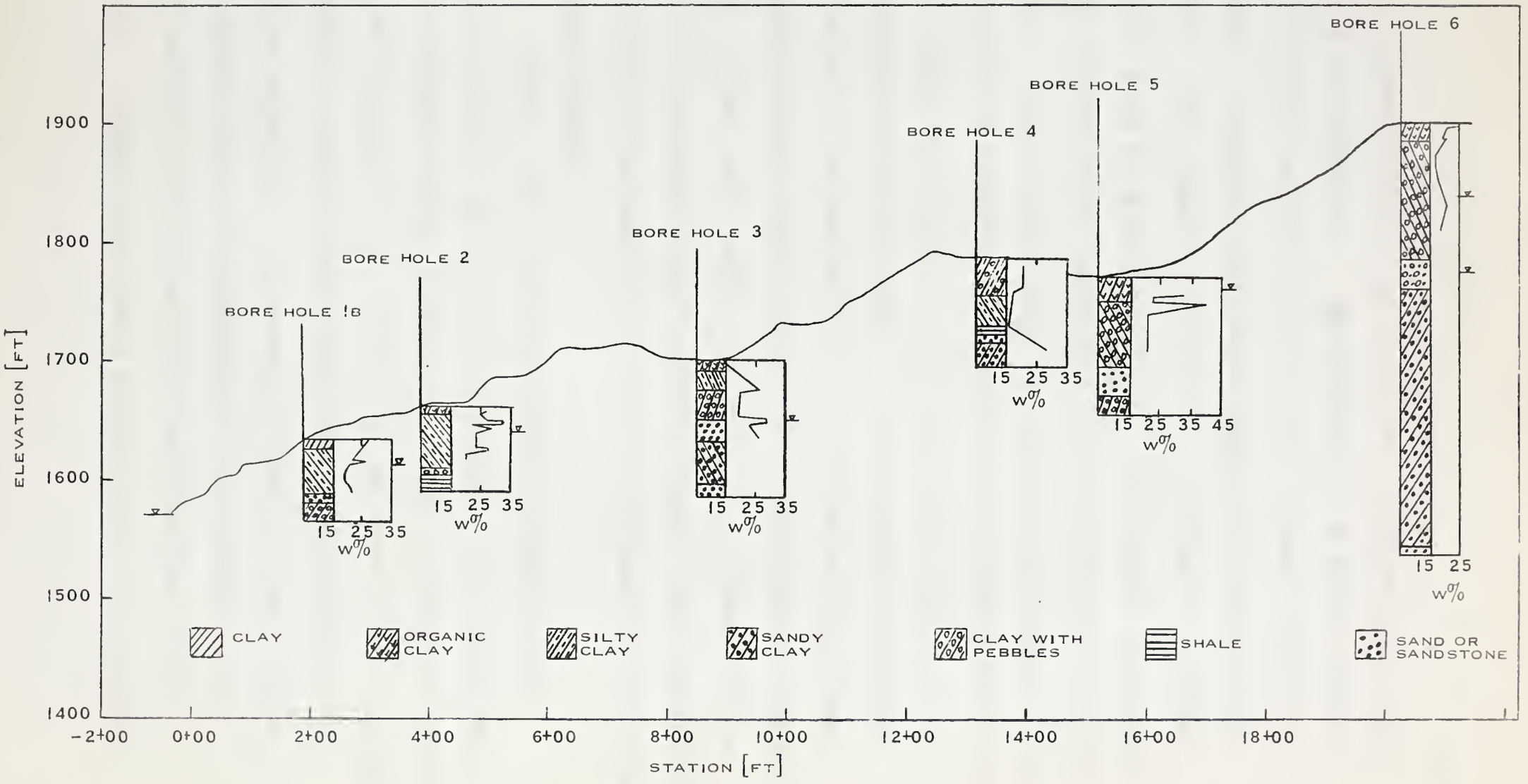


FIGURE 3.1 STRATIGRAPHIC PROFILE ALONG ARC CUT LINE [FIGURE6.1] AT LITTLE SMOKY RIVER

[NOT TO SCALE. LOGGED BY RESEARCH COUNCIL OF ALBERTA, 1960]

A single gravel deposit was found at a comparatively high elevation in borehole 6. In borehole 2, a gravel deposit five feet thick was found above a clay deposit described as clay-shale. A narrow clay-shale seam was discovered in borehole 4. All boreholes showed an increase in the percentage of sand at greater depths. The bedrock clay-shale deposits exhibited no continuity either vertically or horizontally. Both the clay-shale deposits in boreholes 2 and 4 were found at a depth of comparatively low moisture content and the higher stiffness of the samples may have led to a misinterpretation of the data. In the author's opinion, the evidence in the majority of the borehole logs showed that basal gravels were not penetrated although the apparent increase in sand with depth indicated an approach to such deposits. The bedrock was therefore deeper than the greatest penetration of the boreholes since it supposedly was covered with basal gravels.

In the area investigated by Henderson, till was found to occur as a surficial deposit in a large area between Whitemud Creek, Alberta and south to Sturgeon Lake. It was very extensive as a surficial deposit in the Sturgeon Lake area but north of Whitemud Creek it was covered by lacustrine deposits. Henderson pointed out that doughnut shaped mounds were the predominant till feature in the eastern portion of the area which included the Little Smoky site. These mounds were symmetrical with a central,

swampy depression. Henderson estimated that they were 4 to 15 feet in height with a diameter varying from 250 to 450 feet. The mounds appear to be present in the vicinity of the Little Smoky River bridge site as shown on the aerial photograph in Figure 3.2. This author did not personally investigate the mounds around the bridge site to ascertain if they were similar to the ones investigated by Henderson but there is an indication that till is present in the area.

Boreholes at Falher, Alberta, have shown a till thickness of 250 feet and other observations in the area indicated that the till thinned toward the south where at Valleyview, a thickness of four feet was observed by Henderson (1958). Boreholes 3, 4, 5 and 6 in Figure 3.1 exceeded a depth of 75 feet and should, in all probability, have penetrated the till but there was no difinite evidence of shale bedrock in the logs except in boreholes 2 and 4.

Three till sheets were found by Henderson which indicated that there were three separate ice invastions. The youngest till sheet, called the Upper Till, was found at New Fish Creek and areas north and east of it. It contained very few stones, approximately 47 per cent finer than .0025 mm and was grey to grey-brown in colour. The high proportion of clay sizes in this till was attributed to lacustrine clays over which the last glacier advanced. The Upper Till was differentiated from existing lacustrine clays by the strati-

FIGURE 3.2 AIR PHOTO ILLUSTRATING DONUT-SHAPED TILL MOUNDS AT THE LITTLE SMOKY BRIDGE SITE

fication of the latter. The Middle Till, located under the Upper Till, was a brown colour and had a blocky, prismatic fracture pattern as it dried. It had 24 per cent finer than .0025 mm, contained more gravels than the Upper Till and the gravels were predominantly of Canadian Shield origin. The clayey matrix of this till had its origin in the bedrock shales. The Lower Till had more pebbles than the Middle Till, and had a yellow-brown to brownish-grey colour. Of the three tills, the Middle Till had the greatest exposures in the area investigated by Henderson.

Grain-size analysis of samples taken from the Little Smoky site revealed 42 per cent finer than .002 mm and a well-graded grain-size distribution. The colour of the samples was dark-grey and they were not stratified. Some pebbles taken from the samples were identified as quartzites and were of Rocky Mountain origin (Westgate, 1965). The evidence stated above closely agreed with Henderson's description of the Upper Till formation. If this was the case and Middle and Lower Tills were present, then the bedrock formation was considerably deeper than 35 to 40 feet, the depth at which samples were obtained in borehole 3.

Glacio-lacustrine deposits covered the area investigated north of Whitemud Creek, and along the Little Smoky River Valley. These deposits were laid down in two proglacial lakes that formed as the ice receded. Glacial Lake Valleyview covered the area of Sturgeon Lake and

areas northeast along the Little Smoky River. The lacustrine sediments deposited were described as silts overlying massive, laminated clays. The deposit was approximately 10 feet thick. Glacial Lake Falher was formed by the ice-dam to the north and the uplands to the east and west. Clays, silts and sands were carried into this lake by the Peace, Smoky and Little Smoky Rivers and so the sediments had Cordilleran origin. These sediments were deposited over a large till sheet which extended east and west of the Smoky River. The maximum thickness of this lacustrine deposit was about 12 feet. The deposit graded from the clay to sand from the underlying till to the surface of the deposit. At the present time, periglacial sand dunes cap the deposit in the area around the junction of the Smoky and Little Smoky Rivers.

In the vicinity of the Little Smoky bridge site, the lacustrine cover appeared to be silt that was deposited in proglacial lake Valleyview. The cover was thin since the till mounds showed through fairly clearly.

Henderson postulated a typical soil profile of the Little Smoky River from its mouth to the bridge site. The profile showed till overlying the bedrock, the middle till having the greatest thickness of the three mentioned, and lacustrine sediments of clay and silt overlying the till. The till thinned out considerably over the flat terrain beyond the valley since the preglacial valleys received greater thicknesses of till drift than the table land between

valleys.

Character of Tills

Tills are a direct glacial deposit and as such contain a wide range of grain-sizes (Flint, 1963). A till deposit may also have pockets of silt and sand and some bedding of coarser sizes may be found between two till sheets (Elson, 1960). Tills usually exhibit no stratification (Flint, 1963) which infers that no horizontal planes of weakness should be present, such as found in many clay shales. According to Elson (1960) till may also have a jointed structure and oblique joints have been found to be common in clayey tills.

It is evident that tills can be very erratic by nature and this may also be the case with respect to their engineering properties. For example, the "boulder clay" at Selset (Skempton, 1964) showed a low value of cohesion although apparently subjected to large loads in its geologic past.

Summary

In this chapter, the geology of the Peace River area was briefly described. The Pleistocene deposits expected to be found in the Little Smoky River Valley were outlined and mention was made of the fact that the pre-glacial river valley was infilled to a large extent by tills. Slip surfaces at depths less than about 75 feet would in

all probability be located in the tills since it was estimated that the bedrock was somewhat deeper.

CHAPTER 4

APPARATUS AND PROCEDURES

Aim of Test Program

The aim of the test program was to reassess the embankment instability at the Little Smoky River bridge site using residual strength parameters obtained from a direct shear box. The peak parameters were compared to those obtained from consolidated undrained triaxial tests with pore pressure measurements in order to assess the accuracy of the direct shear apparatus and associated procedures. Consolidated undrained triaxial tests with pore pressure measurement on normally consolidated remoulded specimens were carried out to investigate the validity of Skempton's hypothesis that normally and overconsolidated clays had the same residual strengths at the same normal effective stresses, as applied to the Little Smoky Clay Till.

Direct Shear Box

The shear box was constructed for double direct shear for samples having the approximate dimensions of 2 in. x 4 in. x 3 in. The shear box was placed in a tank which was filled with water to ensure no evaporation of

moisture from the sample in a long term test. The centre section of the shear box was displaced by pistons that were guided through teflon bushings at the ends of the tank. The normal load was applied through a lever system and hangar that have a multiplication factor of ten. The lever arm was maintained in a horizontal position by adjusting a screw jack as volume changes occurred. The upper section of the shear box was held in position by set screws. Teflon guides attached to the centre section maintained the alignment of that section while being displaced. The bottom section was fixed in position by shear pins set into the bottom of the tank. The shear force was applied by a constant speed motor driven worm gear. Many different displacement rates could be obtained through an available system of gears. All metal parts in the tank and shear box were of stainless steel. Drawings of the apparatus are at Appendix A and a sketch of the shear box is at Figure 4.1.

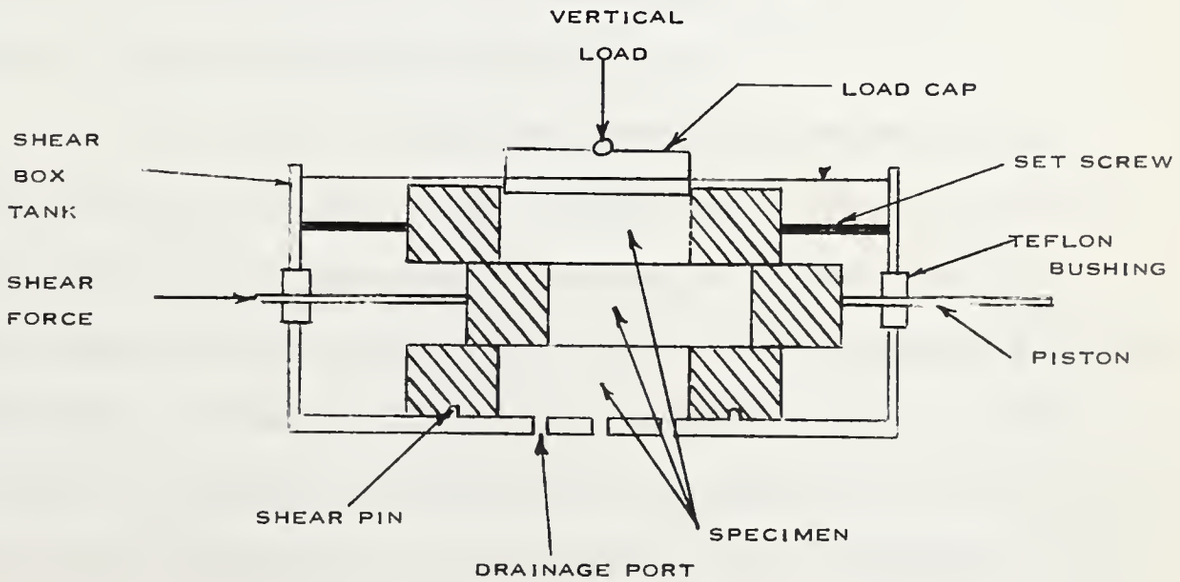


FIGURE 4.1 SKETCH OF SHEAR BOX

It was originally hoped that 6-inch diameter samples would be available and the box was therefore made five inches long. Only 4 3/4-inch diameter samples could be obtained and the box length was then shortened to four inches by shims. These shims can be removed if it is desired to test specimens five inches in length in a future program.

Instrumentation

It was expected from calculations of the time to failure, to ensure a fully drained test in the direct shear box, that a test duration of two or three weeks would be required. In order to facilitate taking readings over a 24-hour period throughout the duration of the test, data was recorded automatically on a digital recorder at specified time intervals. The data was electronically measured by a load cell (shear force) and two DTDT's (Differential Transformer Displacement Transducers), one each for the vertical and horizontal deflections.

The type of load cell used was essentially a metal cylinder in which the strain under axial loading was picked up by attached electric SR4 strain gauges. The change in the resistance of the strain gauges with the deflection of the load cell was recorded as a voltage change by a digital recorder or a voltmeter. The load cell was constructed of duraluminum for a maximum load

of 500 lbs. The area required was 0.017 in^2 and the cell was a hollow tube with an internal diameter of 0.3 inches and a wall thickness of 0.017 inches. The hollow tube design provided sufficient safety against buckling failure and provided sufficient surface area to attach the SR 4 gauges. Design criteria were obtained from the Alcoa Structural Handbook (1960).

The characteristics of the load cell were:

a. Sensitivity: A full Wheatstone Bridge arrangement was used. Two dummy SR 4 gauges were attached to a separate piece of duraluminum and two active gauges were attached to the load cell. This arrangement gave twice the sensitivity of a half bridge. With a Budd strain indicator, the calibration factors was 0.0997 lb per micro-inch, whereas with the digital recorder, the factor was 0.416 lb per millivolt x 10^{-2}. Since the Budd recorder measured to the nearest micro-inch per inch and the digital recorder to the nearest millivolt x 10^{-2}, it was apparent that the sensitivity using the digital recorder was approximately 25 per cent of that of the Budd recorder. The full scale error of the digital recorder was determined to be ±0.07 per cent. The full scale reading was 12 millivolts for the 500 lb load, and the error was, therefore, ±0.35 lb.

b. Temperature: Although the full bridge arrangement

was temperature compensated with the dummy gauges, it was found during creep tests that temperature variations did affect the load cell. The temperature deviation was estimated to be ±0.15 lb/°F.

c. Creep: A three day creep test was performed on the load cell under a load of 132 lbs. According to Holubec and Scott (1965), a linear relationship exists between the logarithm of time and creep. Creep was 0.8 lbs in 1000 minutes which, when extrapolated to 16 days (approximate test duration), would result in a creep of 1.2 lbs. Creep may occur in the load cell and in the bonding cement (Holubec and Scott, 1965). Creep in the load cell increases the apparent load and creep in the bonding cement decreases the apparent load. It was interesting to note that the load apparently decreased in the creep tests conducted indicating more creep in the cement than in the load cell. The cement creep can be reduced by decreasing the thickness of bonding cement (Holubec and Scott, 1965).

The vertical and horizontal deflections in the direct shear test were measured by Differential Transformer Displacement Transducers. This transducer consisted of a transformer and a rod which was displaced between the primary and secondary coils. The primary was DC excited and the DC output varied with the rod displacement. The relationship between the rod displacement and DC output was linear over the recommended displacement range of the

manufacturer. There was no hysteresis and the deviation from the linear was 0.5 per cent of full-scale reading.

The DTDT's showed some variation with temperature. This was not considered to be too serious with the horizontal deflection. Accurate readings of the vertical deflection were required since considerable scatter of points occurred near the end of primary consolidation in the direct shear box and it was difficult to determine when 100 per cent consolidation had been achieved. Accurate vertical deflection readings were also required to assess the volume changes during shear. Temperature was therefore recorded throughout the duration of the direct shear tests by means of a thermocouple. The variation in vertical deflection with temperature was found to be 7×10^{-4} in/°F and this calibration would likely vary for different DTDT's.

The load cell and two DTDT's were connected to a digital recorder. A DC power supply converter provided excitation for the instruments and converted the AC signals to DC for the input to the recorder. On a command given by the digital clock, the voltage at each channel in use was measured by a voltmeter and values were recorded on a printed tape. Readings could be taken at any time manually or up to hourly intervals automatically. A typical set of readings is illustrated at Appendix B. It is worth noting that with an additional modification, it would be possible to record readings directly on IBM punch cards.

With a temperature variation of ±2°C, the full scale error of the recorder was approximately ±0.07 per cent of full scale.

Calibration Direct Shear Box

Before starting the test program, the values of piston friction, the normal tare load and the lever multiplication factor had to be determined. The results of this calibration were:

a. Friction: The two pistons passed through teflon bushings at either end of the shear box tank. With the two pistons inserted and the empty shear box in place, the friction of this system was measured with a calibrated load cell. The load increased to a constant maximum value of 8.23 lbs. The friction of the two pistons without the shear box in place was 1.18 lbs. Prior to an actual test with a soil specimen in the shear box, the shear box sections were separated slightly. It should then be expected that only the two pistons would offer frictional resistance. It was difficult to determine during a test whether the shear box sections were separated entirely and it was therefore likely that the frictional resistance of the system was somewhere between 1.18 and 8.23 lbs., but probably closer to the lesser value.

b. Normal Tare Load: The determination of the normal tare load was complicated by the lever system. It was decided to obtain the tare load by a proving ring inserted

under the hangar with the shear box and tank removed. The average of readings from three different calibrated proving rings gave a normal tare load of 98.67 lbs. To this was added the weight of the loading cap and stainless steel teeth giving a load of 101.26 lbs. Since different types of porous stone and sizes of load ball bearings may be used, the weights of these would have to be added prior to each test. For a specimen area of 51.6 cm^2, the normal tare pressure is 0.914 kg/cm^2.

c. Lever Ratio: The lever system was designed to provide a multiplication factor of 10. Using a proving ring as described above, and various loads on the pan, the 10:1 ratio was consistently attained within ±0.2 per cent.

d. Available Test Rates: A constant speed (24 rpm) electric motor provided a constant rate of deformation. Reductions in rate were provided by a 25 speed gear box, a 16:62 ratio from drive sprocket to the gear box that housed the worm and a reduction of 50:1 in the worm gear box. The rates varied from 17,200 $\times 10^{-6}$ inches per minute to 7×10^{-6} inches per minute, at the worm. Two rates were calibrated and they agreed within ± 0.5 per cent of the manufacturer's rating .

The characteristics of the direct shear box and accompanying instrumentation are summarized in Table 4.1.

TABLE 4.1

CHARACTERISTICS OF DIRECT SHEAR BOX
AND ASSOCIATED INSTRUMENTATION

1.	LOAD CELL	Constant	0.416 lbs/mv x 10^{-2} ±4%
		Recorder Accuracy	±0.07% Full Scale
		Temperature Error	0.15 lbs/°F
		Creep	0.8 lbs in 1000 min. (Negative)
2.	DTDT	Constants	174 x 10^{-6} in/mv (Vertical Deflection) 238 x 10^{-6} in/mv (Horizontal Deflection)
		Deviation From Linear	±0.5% Full Scale
		Temperature Error	±7 x 10^{-4} in/°F
		Recorder Accuracy	±0.07% Full Scale
3.	DIRECT SHEAR BOX	Friction	1.18 lbs Pistons only 8.23 lbs Pistons and Box
		Normal Tare Load	101.26 lbs ±1.4lbs Hangar, Lever, Load Pan, Loading Cap and Teeth.
		Ratio Pan Load: Specimen Load	1:10 ±0.2%
		Rates of Strain	17,200 x 10^{-6} to 7 x 10^{-6} in/min. ±0.5%

Procedure

The detailed procedure for the direct shear box is at Appendix C. Modifications and important considerations in the procedure used in this program were:

a. Trimming: It was found that a medium stiff clay till could be cut with a wire saw using the trimmer (Appendix A). There were numerous small pebbles in the Little Smoky Clay Till which made trimming difficult and time consuming. When the cutting of the wire saw was obstructed by a pebble, the specimen was removed from the trimmer and the pebble carved out of the sample. Cavities left by the pebbles were patched with trimmings. Final trimming to obtain a precise fit in the shear box was done with a sharp knife with the specimen out of the trimmer. Approximately two hours were required to trim a sample.

b. Moisture Contents: A series of initial and final moisture contents were taken in order to determine the changes in moisture content along the failure planes and vertically through the specimen. The initial moisture contents naturally had to be taken outside the specimen but were taken at approximately the same elevations as the final moisture contents. The moisture content profiles along the failure planes indicated the variation in shear stresses along the failure plane and the variation in normal load. The vertical profile was taken in order to determine the migration of moisture during shear and in particular to

determine if there was a migration toward the failure planes as found by Skempton (1964).

c. Time to Failure: (1) It was necessary to determine a rate of testing to ensure that a fully drained test was being conducted and that stresses were effective. It was initially thought that a suitable time to failure would be determined by trial and error but this would have involved a minimum of three tests at one normal pressure and the time available did not allow for this. A minimum of three additional tests at different normal pressures would also be required to obtain the residual strength parameters. As a result it was decided to calculate the time to failure using a formula developed by Gibson and Henkel (1954). The comparison of the strengths obtained with those from the triaxial tests would probably indicate whether fully drained tests were conducted.

(2) Gibson and Henkel (1954) developed a formula to compute the time to failure for drained direct shear tests from the general theory of consolidation. Employing this theory and a relationship between volume change and the change in effective stresses derived by Skempton (1954), the basic differential equation obtained was

$$c_b \frac{\delta^2 u}{\delta^2 Z} - \frac{\delta u}{\delta t} = - \frac{\delta u_o}{\delta t}$$

where c_b was the coefficient of consolidation obtained from consolidation in the direct shear box, u was undissipated excess pore pressure and u_o was the pore pressure that would arise in an undrained test subjected to the same stress conditions. The solution of the differential equation gave the time to failure $t_f = \frac{H^2}{2\ c_b\ (1 - u_c)}$ where H^2 was the sample height and u_c was the degree of consolidation desired at failure. In the development of this formula, the following assumptions were made:

a. the assumptions applicable to the theory of consolidation,

b. the development was based on a normally consolidated clay. Gibson and Henkel (1954) state with respect to overconsolidated clays, that the formula "provides a measure of the extent to which conditions of the test approximate to the fully drained condition."

c. the pore pressure at any point in the specimen was dependent only on the change in principal stresses and not their rotation,

d. the failure zone was subjected to a uniformly distributed shear stress,

e. the deformations were uniform throughout the full depth of the specimen,

f. the undrained pore pressure u_o, increased linearly with strain to a constant peak value from where

it dissipated with time,

g. the coefficient of consolidation c_b determined from the direct shear box was equal to the coefficient obtained in a consolidometer for the same soil type.

(3) Skempton (1964) pointed out that the clay particles were very heavily oriented in a narrow zone adjacent to the failure plane. The soil in this narrow zone was also found to be at a higher moisture content than the rest of the specimen at the end of the test. Hvorslev (1960) found that deformations were very much higher adjacent to the shear planes than elsewhere in direct shear. These facts indicated that deformations in a direct shear test were certainly not uniform as assumed by Gibson and Henkel (1954) with the result that the time to failure calculated by the formula was likely on the side of safety, especially for stiff soils. An attempt was made to determine the distance from the failure plane in which severe particle orientation occurred by means of investigating thin sections subjected to plane polarized light. An examination of vertical moisture content profiles assisted in determining the thickness of the zone adjacent to the failure planes that underwent deformation. It is likely that the thickness of the zone of deformation adjacent to the failure plane can be considered as the length of the shortest drainage path in drained tests.

(4) After an estimate of the time to failure was made, the deflection at failure was approximated. For the Little Smoky Clay Till a deflection of 0.1 inch was estimated as the failure deflection and this proved to be the deflection at failure in the tests conducted. The rate of strain then selected was calculated to be 6.9×10^{-5} in/min.

Triaxial Testing

Consolidated undrained triaxial tests with pore pressure measurements were conducted on undisturbed and remoulded specimens of the Little Smoky Clay Till. Tests were conducted at different cell pressures to obtain a Mohr strength envelope in terms of effective stresses.

The Geonor triaxial apparatus was used for the triaxial tests (Bjerrum, DiBiago and Kjaernsli, 1957). A transducer was employed to measure the pore pressures developed during the test. The transducer was inserted into a stainless steel manifold containing a bleed value and the manifold was connected to one of the pore pressure outlets at the base of the triaxial cells using existing Geonor adaptors. A calibration curve for the 100 psi transducer used is at Appendix D. The transducer eliminated the necessity of continuous nulling of the Geonor pore pressure indicator. Voltage changes of the transducer were recorded on a Baldwin strain indicator.

Back pressure was applied to the undisturbed specimens after the consolidation stage was completed in order to saturate the specimen. The back pressure used was in accordance with that recommended by Bishop and Henkel (1962) except that the sum of the cell pressure and back pressure was never greater than 6 kg/cm^2 which could not be exceeded for tests of long duration using the Geonor equipment. The back pressure was applied through the copper tubing (pore pressure lead in normal testing) into one of two ports leading to the pedestal. The transducer was connected to the second port. The back pressure was applied after normal working hours and left overnight in order that equalization could occur. Shear tests were started the following morning. Back pressures were not used for tests conducted on remoulded specimens since the degrees of saturation were approximately 100 per cent and negative pore pressures were not expected to occur.

The principal concern in obtaining a suitable time to failure for a stiff clay was the equalization of pore pressure. Bishop and Henkel (1962) mention that a stiff clay specimen may deform in comparatively thin zones indicating that a much slower rate of testing than normally used is required in order that the correct pore pressure at

failure is measured at the base of the specimen. There is evidence that five days may be required for specimens three inches in height. The coefficient of consolidation obtained in triaxial consolidation of the undisturbed specimens varied from 1.5 to 3 x 10^{-4} cm^2/sec. According to Bishop and Henkel (1962), equalization at failure may be obtained in one working day for specimens having this coefficient, if side drains are used. Calculated failure times can vary from 1.2 hours to 28 hours depending on the estimate of drainage efficiency. It was decided to test at a strain rate of 0.11% per hour with failure estimated at 2.5% strain. This rate was much slower than was previously used on Little Smoky Clay Till (Brooker, 1965), which provided an opportunity to investigate the effects of test duration on the strength of this soil type.

Summary

In this chapter, the apparatus and instrumentation to be used in the test program were described briefly. Calibration constants and the expected accuracy of the equipment were discussed and summarized in Table 4.1. Special points regarding the procedures to be used were mentioned including the methods of determining the time to failure for fully drained direct shear tests and consolidated undrained triaxial tests with pore pressure measurements.

CHAPTER 5

RESULTS AND DISCUSSION

Soil Properties

The borehole logs for ARC3 (1960) and 3A (1965) are at Appendix E. Borehole ARC3A was drilled by the Department of Highways of Alberta with the author supervising. With the aid of a technician, who had previously logged ARC3, and site plans available at the Department of Highways, the location of borehole ARC3A was estimated to be within 75 feet of borehole ARC3. Reasonably accurate location of the borehole was essential if strength results were to be comparable.

Dry drilling was used for the full penetration of ARC3A and samples were taken to establish profiles of Atterberg Limits and moisture content with depth. Three penetration tests were conducted and three unconfined compression tests were carried out from the available samples. Continuous sampling was undertaken from 35 feet to 44 feet using the 4.75-inch diameter Shelby tubes provided by the Department of Highways. This depth corresponded with ARC3 samples U-9 to U-13 of which samples U-10 and U-12 were used for strength tests conducted in 1960.

Table 5.1 illustrates the soil properties obtained

from the large Shelby samples. Three values are shown for Atterberg Limits, grain size and specific gravity representing samples 4, 1 and 3 respectively. Since the difference between these values was small, they were averaged for the calculation of activity number and for plotting on the Plasticity Chart. It should be noted that the sum of the milliequivalents per 100 grams air dried soil (me/100 gm a.d.s.) representing individual cations did not agree with the total cation exchange capacity. The me/100 gm a.d.s. of the calcium cations was high and the discrepancy was due to the fact the pore water contained an abundance of calcium carbonate (Lowe, 1965).

Figure 5.1 is a plasticity chart illustrating the range of 1960 test results and the location of the 1965 value. The group of values tended to fall above and parallel to the 'A' line. The 1965 test result appeared to have slightly more plasticity than the mean of the 1960 tests but fell within the indicated range.

Figure 5.2 illustrates the grain-size distribution range of the 1960 test results and the range for the 1965 tests, which was obtained from three trials. A comparatively greater percentage of clay sizes was associated with the 1965 result which likely accounted for its slightly higher plasticity. Dispersion time and the amount and concentration of dispersing agent used in the wet mechanical analysis may have also contributed to the apparent discrepancy in the

TABLE 5.1

PROPERTIES OF LITTLE SMOKY CLAY TILL

PROPERTY	TEST VALUES			AVERAGE
Specific Gravity	2.71	2.69	2.71[a]	2.71
Natural Water Content				18.3%
Liquid Limit	48.5%	48.7%	47%	48.1%
Plastic Limit	16.7%	17.0%	16.8%	16.8%
Shrinkage Limit	10.8%	10.3%	11.3%	10.8%
Plastic Index	31.8%	31.7%	30.2%	31.2%
Liquidity Index				0.05
Grain-Size Distribution[b]				
% Sand	23	24	24	24
% Silt	35	34	34	34
% Clay	42	42	42	42
Activity Number				0.74
Clay Minerology[c]				
Montmorillonite				58%
Illite				18%
Chlorite				12%
Quartz				12%
Cation Exchange Capacity				17 me/100 gms a.d.s.
Sodium				1.1 me/100 gms a.d.s.
Calcium				35.3 me/100 gms a.d.s.
Magnesium				3.0 me/100 gms a.d.s.
Potassium				1.1 me/100 gms a.d.s.
Permeability[d]				3×10^{-8} cm/sec @ e = 0.5

(a) Determined from samples 4, 1 and 3 respectively.
(b) MIT Scale
(c) Determined from clay fraction samples 4, 1 and 3.
(d) Determined from one dimensional consolidation tests on undisturbed specimens.

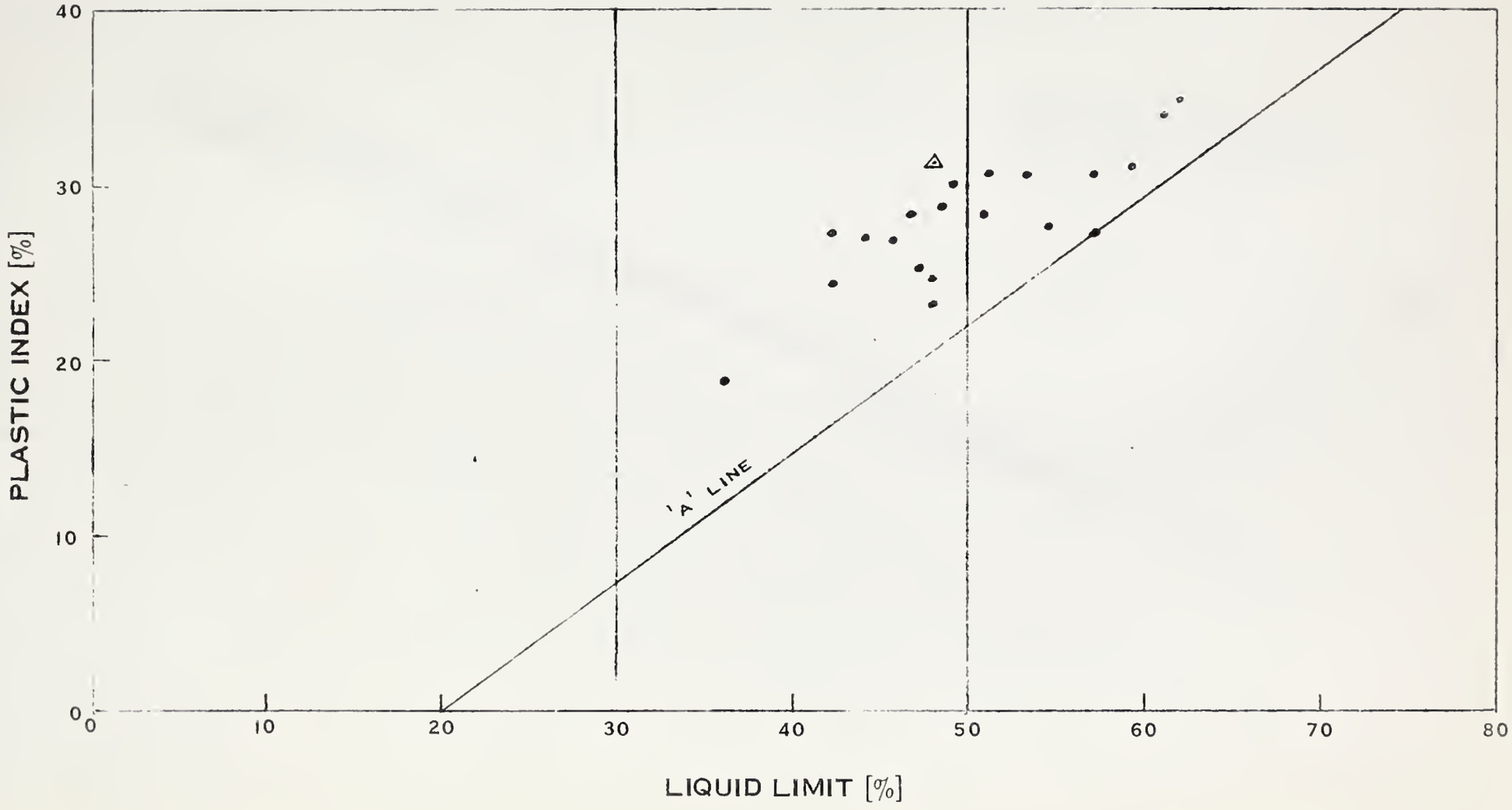


FIGURE 5.1 PLASTICITY CHART FOR LITTLE SMOKY CLAY TILL, 1962 • , 1965 △

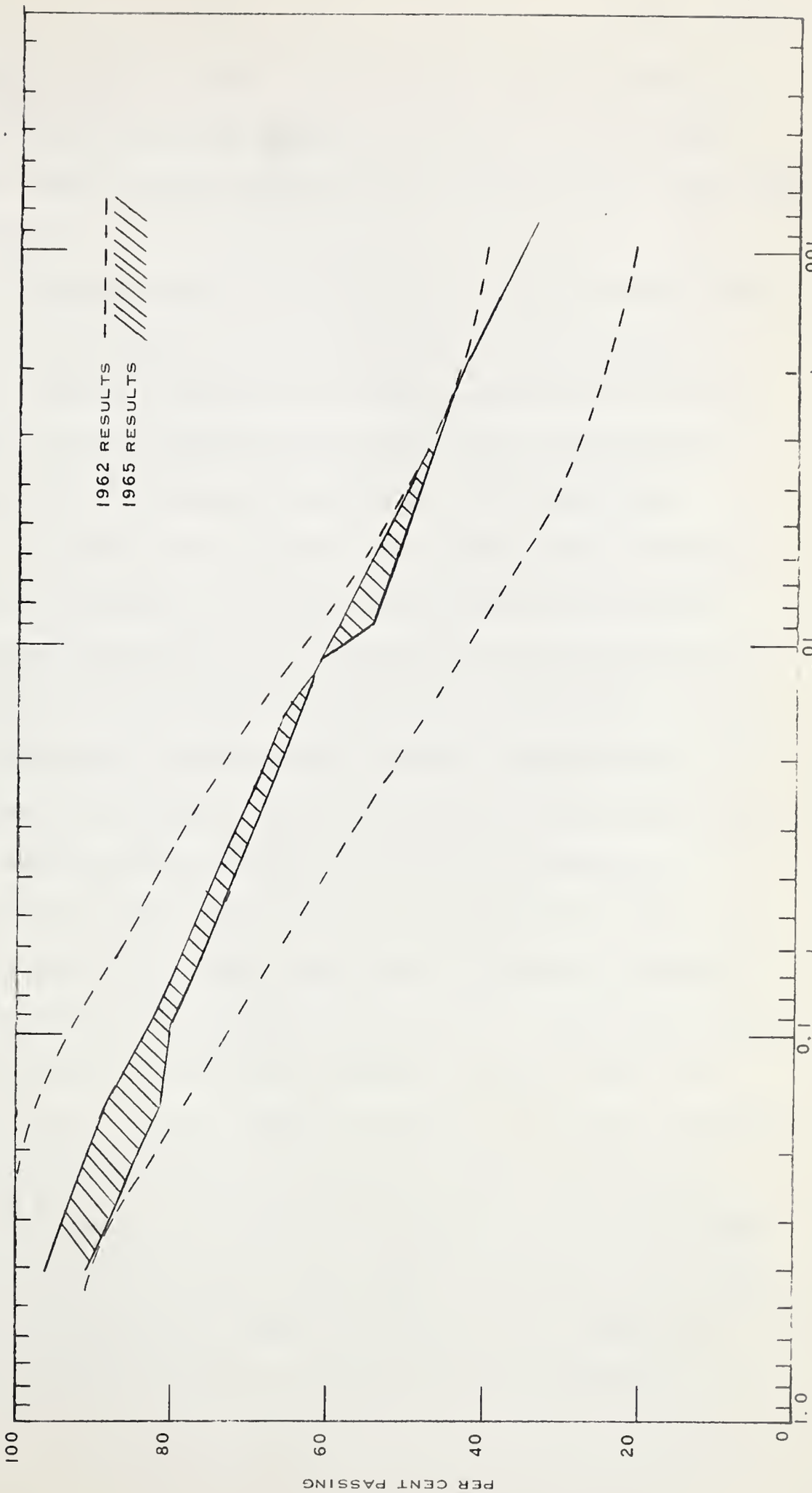


FIGURE 5.2 GRAIN SIZE DISTRIBUTION FOR LITTLE SMOKY CLAY TILL

percentages of clay sizes. All the test results showed a tendency toward equal percentages of clay, silt and sand sizes (MIT scale) indicating that till was the soil type under consideration.

Typical consolidation e log P curves obtained from the undisturbed samples from ARC3A are shown in Appendix F. The e log P curves indicated free swell pressures of from 0.57 kg/cm^2 to 0.93 kg/cm^2 and a rebound to approximately 90 per cent of the initial void ratio. The swelling pressures recorded were a little lower than those obtained previously for samples U-9 and U-10 of ARC3 (0.85 kg/cm^2 and 1.5 kg/cm^2, Appendix E). It may be of some concern as to the reason for such high rebound in a clay where calcium was the predominant exchangeable cation. High rebound characteristics and substantial swelling pressures have been generally attributed to sodium soils and the osmotic pressure effect. Ali (1965) has shown, however, that although sodium soils swell more than calcium at comparatively high moisture contents, the swell is approximately the same at moisture contents slightly greater than the shrinkage limit. In the samples tested, the initial moisture content was one to two per cent higher than the plastic limit or approximately 8 per cent higher than the shrinkage limit. The trend indicated that high rebound and swelling characteristics could be found in clays of low liquidity index with either calcium or sodium as the predominant

exchangeable cation.

From the investigation of residual strength made by Skempton, one of the common characteristics associated with the majority of case histories studied was the fact that the clays were fissured. The author could find no evidence of fissures, joints or cracks in the specimens obtained. In one instance a conical slickensided area was discovered that was approximately perpendicular to the direction of sampling. This appeared to be a shear caused by sampling. Drying tests were also conducted to obtain an indication of stratification but none was observed although small pockets of silt and sand were found in the specimens. In the author's estimation the clay till was intact.

Similar values of natural moisture content, Atterberg Limits, grain-size percentages and swelling pressures were obtained in boreholes 3 and 3A. The description of the soil types reported in the borehole logs was also similar. It was then evident that the soil under consideration was the same as that previously tested to determine the effective shear strength parameters.

Direct Shear Test Data

Table 5.2 illustrates the strength data obtained from four direct shear tests. The peak and residual load recorded by the load cell is shown for each normal stress as well as the deflection to peak and residual strength.

TABLE 5.2

STRENGTH DATA FOR DIRECT SHEAR TESTS

Sample	Normal Stress (kg/cm^2)	Load (lbs)	Deflection (in.)	Peak Shear Stress (kg/cm^2) (a)	Residual Shear Stress (kg/cm^2) (a)	Moisture Content % Initial	From Consolidation Phase (b)	Final
S-6	0.96	168.1	0.05	0.80 (1)	0.53 (1)	18.42	18.19	21.55
		88.1	0.91	0.78 (2)	0.40 (2)			
				0.74 (3)	0.37 (3)			
S-1	1.5	179.7	0.1	0.85	0.45	18.42	18.0	20.33
		84.4	1.37	0.83	0.38			
				0.80	0.35			
S-3	2.1	264.0	0.1	1.25	0.96	17.70	16.84	19.70
		185.8	1.88	1.22	0.86			
				1.19	0.83			
S-6	3.08	326.0	0.09	1.54	1.41	18.21	17.22	19.64
		241.1	1.80	1.50	1.11			
				1.46	1.07			

Note: (a) Reading from top to bottom, stresses include:
(1) Piston friction, full area correction
(2) Piston friction, no area correction
(3) Piston and box friction, no area correction

(b) Calculated from vertical deflection

The deflection included the distance travelled under stress reversals except in the case of the test with normal stress equal to 0.96 kg/cm^2 where no stress reversal was undertaken. The moisture contents were calculated as follows:

a. Initial Moisture Content: represented the average of 32 moisture contents, six taken vertically at each of the four compass points around the specimen and an additional four at each of the likely failure planes. This large number of moisture contents was required to compare final and initial moisture content profiles along the failure planes and vertically through the sample.

b. Consolidation Moisture Content: calculated on the basis of the volume change during the consolidation phase in the direct shear box, assuming saturation.

c. Final Moisture Content: obtained by averaging ten moisture contents taken along the failure planes at the completion of the test.

There was a problem in determining the actual area over which the shear stresses were acting. This was due to the fact that adhesion between the lip of the shear box and the specimen was probably mobilized to some extent. According to Lambe (1962), if the adhesion between the lip and specimen is one-half of the shear stress, then the true shear stress will be obtained by using the initial area of the specimen in calculations. If the lip of the box is very sharp, then the initial area should be corrected by the full

deflection. Since the value of adhesion between the lip and the specimen was unknown for the direct shear tests conducted, the necessary correction to calculate the true area could not be applied.

It was mentioned in Chapter 4 that piston friction only was likely active since box friction was negligible or very low due to the separation of the shear box sections prior to shear. However, the full range of stresses possible were calculated to illustrate the combined effect of friction and area correction on the values of shear stress. For example, in Table 5.2, the highest peak shear stress for a normal stress of 0.96 kg/cm^2 was 0.80 kg/cm^2 (piston friction and full area correction) and the lowest was 0.74 kg/cm^2 (piston and box friction, no area correction). The variation was not large for the peak shear stresses but was much greater for the residual stresses, since large deflections were associated with the latter.

Since the box friction was small or negligible, it was considered that the stresses, calculated from the average of the fully corrected area and the initial area in conjunction with piston friction, would give representative values. Therefore, the first two values of shear stress illustrated in Table 5.2 (0.80 kg/cm^2 and 0.78 kg/cm^2 for normal stress = 0.96 kg/cm^2) were averaged for stress - deflection plots, Mohr diagrams and moisture content - log strength relationships.

Several assumptions made in the development of the formula to calculate the time to failure were not valid. The coefficient of consolidation (C_b) determined from the consolidation phase in the direct shear box was much greater than that determined in the oedometer (C_v) for all but one test (Table 5.3). Calculated failure times were theoretically too short, and if residual negative pore pressures were active in the failure zone at failure, peak strengths were too high. This may have been offset to some extent by the fact that deformations apparently took place in narrow zones and the use of one-half the sample height overestimated the time to failure. Furthermore, the shear stresses on the failure plane were not uniform, as indicated by the variation in moisture content, which in one case was about 6 1/2 per cent (Figure 5.9). There was then some doubt about the validity of the formula for calculating the time to failure and it is advisable to use a trial and error procedure in future programs.

Figures 5.3, 5.4 and 5.5 illustrate stress-deflection curves for four direct shear tests conducted. The break in the stress-deflection curves represented a stress reversal. For example, in the test at a normal load = 3.08 kg/cm^2, the direction of shear was reversed three times. The time lapse for carrying out the stress reversal was no longer than 20 minutes in all cases.

TABLE 5.3

CONSOLIDATION DATA
DIRECT SHEAR TESTS

Sample	Normal Stress	Coefficient of Consolidation Shear Box	Coefficient of Consolidation Oedometer
	kg/cm^2	$(\frac{in^2}{min} \times 10^{-4})$	$(\frac{in^2}{min} \times 10^{-4})$
S-6	0.96	18.3(1) 18.0(2)	11
S-1	1.5	156.9 389	10
S-3	2.1	114.4 212.7	10
S-6	3.08	2190 4900	12

Notes: (1) Log fitting method
(2) Square root time method

Although the results were fairly scattered, there was a definite trend toward a constant ultimate strength being achieved after failure had occurred. However, the selection of the value of residual strength was difficult, especially after one or two stress reversals since the trend in shear stress changed with each stress reversal. The author, therefore, selected the lowest value of residual strength recorded to obtain the shear strength parameters.

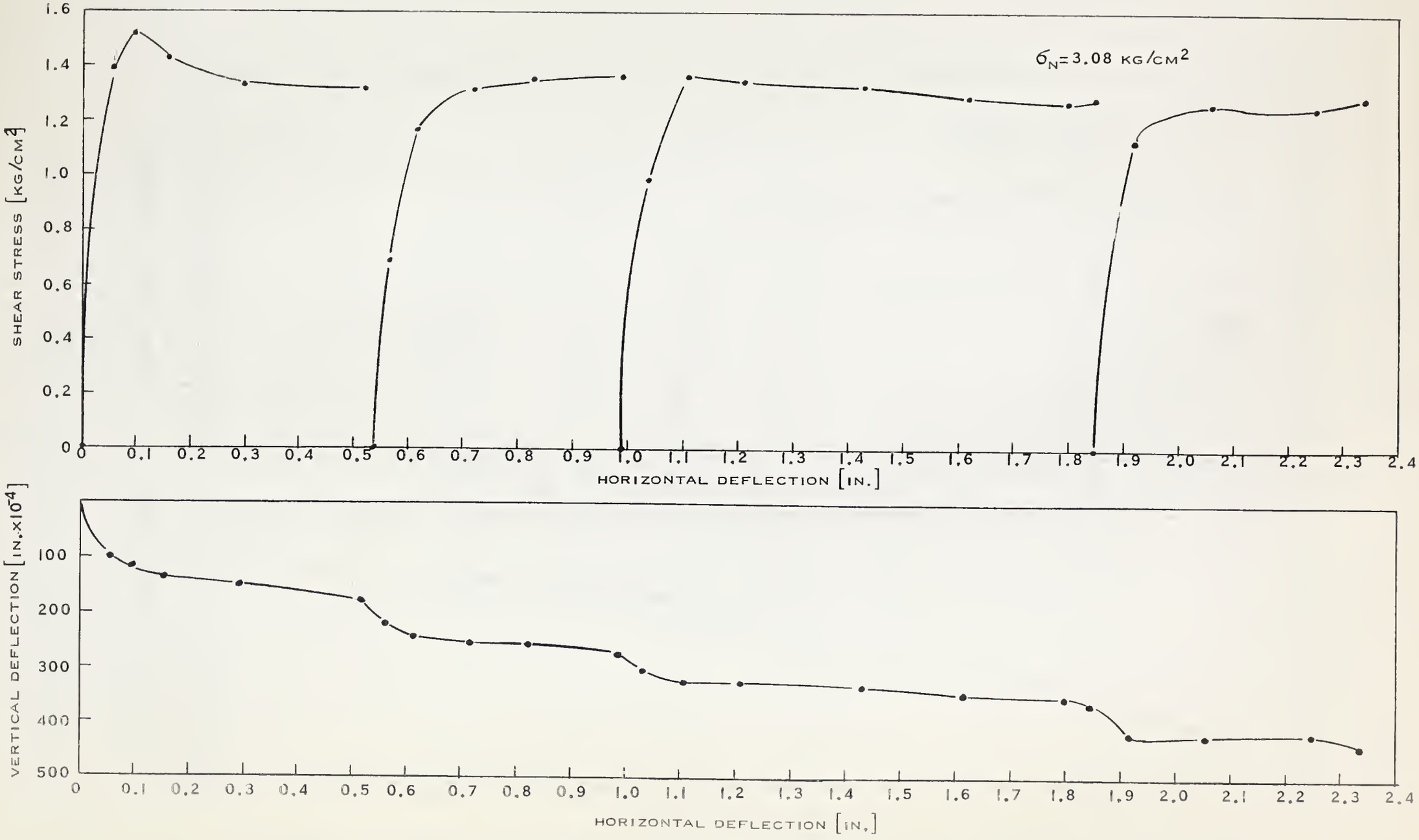


FIGURE 5.3 SHEAR STRESS AND VERTICAL DEFLECTION VS. HORIZONTAL DEFLECTION IN DIRECT SHEAR FOR LITTLE SMOKY CLAY TILL

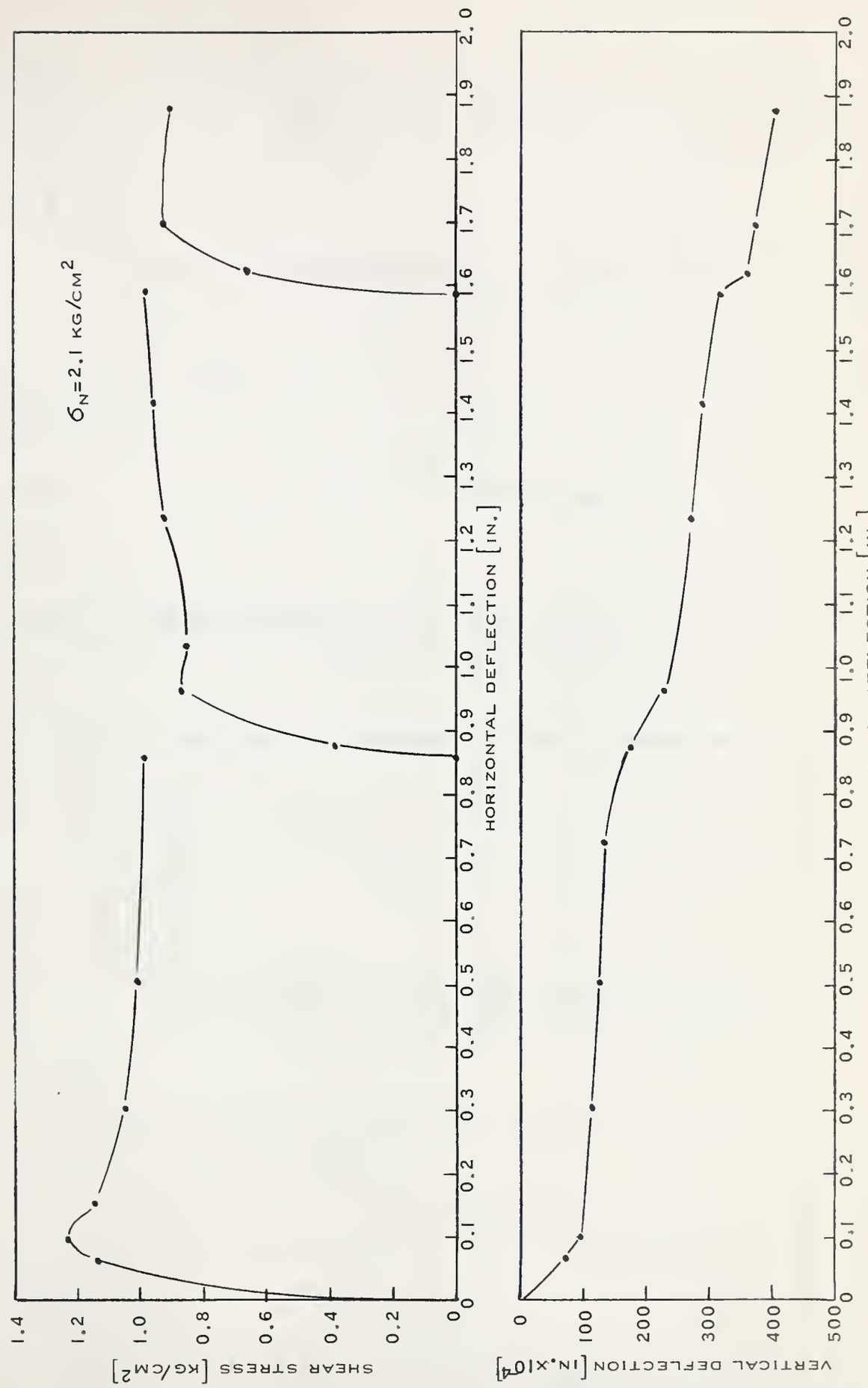
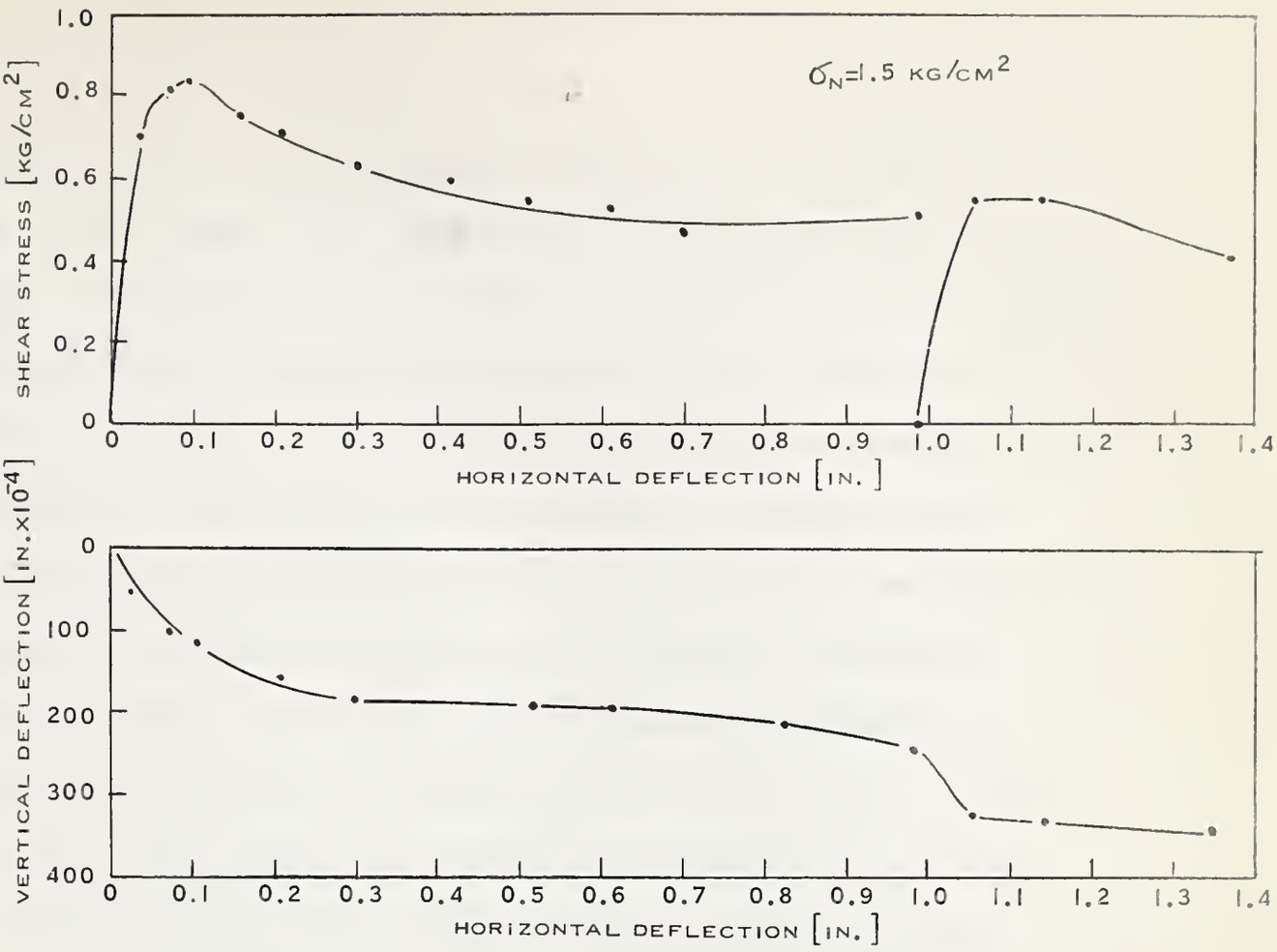


FIGURE 5.4 SHEAR STRESS AND VERTICAL DEFLECTION VS. HORIZONTAL DEFLECTION IN DIRECT SHEAR FOR LITTLE SMOKY CLAY TILL



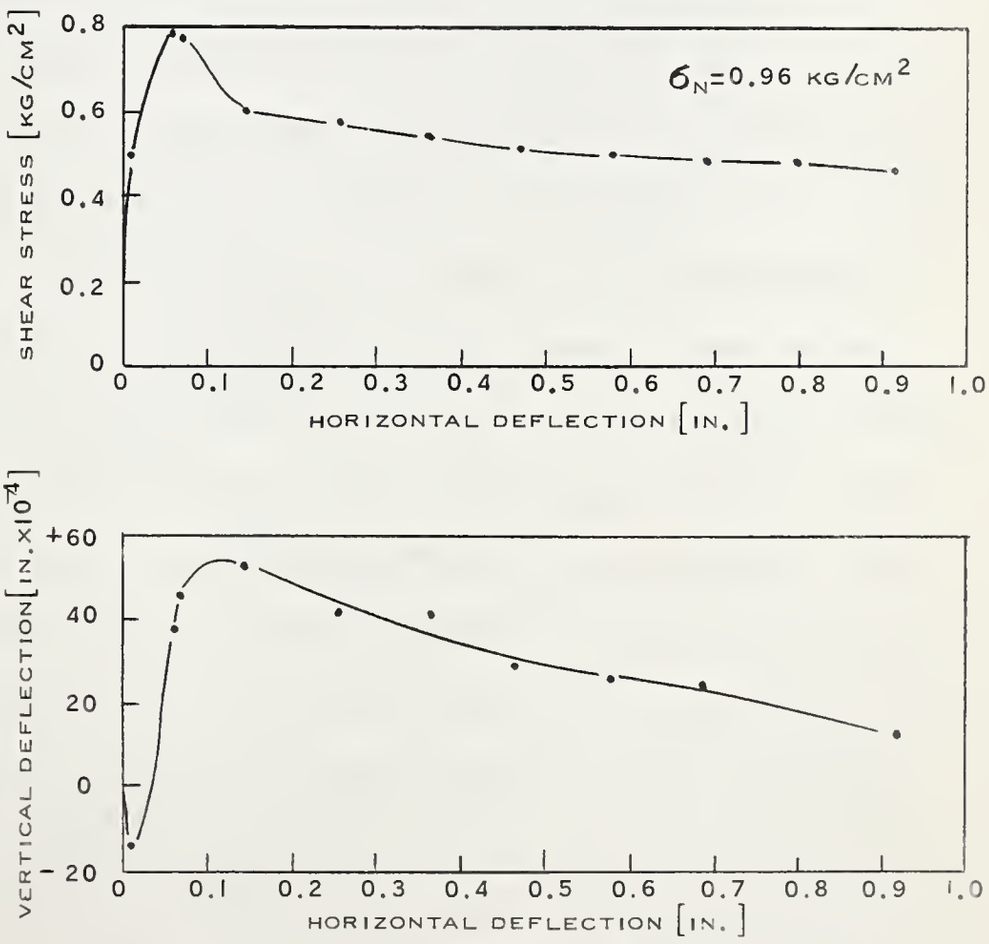


FIGURE 5.5 SHEAR STRESS AND VERTICAL DEFLECTION VS. HORIZONTAL DEFLECTION IN DIRECT SHEAR FOR LITTLE SMOKY CLAY TILL

The figures show that reversing the direction of shear was certainly not ideal and, as a result, the selection of the residual strength value was not clear cut. The centre portion of the box could be pushed approximately one inch in one direction and in the tests conducted, it was estimated that approximately 80 per cent of the drop in strength after failure occurred at a deflection of one inch in one direction. It appeared feasible then, that satisfactory values of residual strength could have been obtained by shearing in one direction only out to one inch and applying a suitable correction. This would have also eliminated the probable effect of partially destroying a soil structure already established during shear in the previous direction.

A further observation from Figures 5.3, 5.4 and 5.5 was that a stress reversal was accompanied by a decrease in the volume of the specimen. Evidence from moisture content profiles indicated that dilation seemed to be occurring adjacent to the failure planes during shear. The author was then of the opinion that a stress reversal would result in additional consolidation. If a clay particle was being forced up and over an adjacent particle during shear, then it seemed reasonable that an opposing shear force would tend to return the particle to its original position resulting in a volume decrease. Hvorslev (1960) proposed that a stress reversal would decrease the pore pressure but maintain the same sign. Dilation and its associated negative pore

pressures would then have to dissipate to some extent with stress reversal allowing some consolidation to occur.

All tests showed a decrease in volume during the application of shear stresses except for the test at a normal stress equal to 0.96 kg/cm^2 (Figure 5.5). This observation indicated that dilation was not occurring in specimens subjected to higher normal stresses. Since dilation is generally associated with overconsolidated clays, it was reasonable to expect that the Little Smoky Clay Till had a fairly low preconsolidation load.

Triaxial Test Data

The data obtained for consolidated undrained tests with pore pressure measurement is shown in Table 5.4. Six tests were conducted on undisturbed specimens and three on specimens remoulded and consolidated from a moisture content higher than the liquid limit. The initial moisture content was obtained from the trimmings and the consolidation moisture content was calculated from the amount of water lost during consolidation. The final moisture content was the average of three samples taken vertically through the specimens. Approximately a 1/8-inch skin was removed around the specimen to ensure that the final moisture contents were not influenced by water intake during dismantling.

Two pore pressure reactions were attempted on the undisturbed specimens tested at cell pressures of 2.2 and

TABLE 5.4

TRIAXIAL TEST DATA

Sample	σ_3	Back Pressure	B	S_i	S_f	Moisture Content %			At Max. Deviator			Per cent
						Initial	Cons.	Final	$\sigma_1-\sigma_3$	σ'_{3f}	A_f	Strain
	kg/cm^2	kg/cm^2	%	%	%				kg/cm^2	kg/cm^2		
undisturbed												
S-6	1.18	2.18	-	97.1	99.5	18.17	17.43	19.04	1.67	1.31	-0.08	5.98
S-4	1.4	2.2	-	93	96.4	18.21	17.77	19.09	1.83	1.17	+0.13	6.54
S-4	1.4	2.16	-	96	98.4	18.30	17.42	19.70	1.83	1.47	-0.04	6.52
S-6	2.2	2.2	1.10 (5)(a)	97.1	97.5	18.08	17.77	18.56	2.00	1.74	+0.23	3.11
S-4	2.94	2.2	-	95.1	97.5	18.32	17.45	18.26	3.76	2.87	+0.024	8.14
S-4	5.02	0.52	5.0 (5)	98	99.2	17.96	16.12	18.19	4.68	4.06	+0.2	7.86
remoulded												
S-6	1.4	-	82 (10)	100	96.3	41.71	33.71	35.70	0.59	0.52	1.16	9.14
S-6	3.2	-	82 (9)	96.5	93.3	41.90	28.78	30.13	1.81	1.15	1.14	9.40
S-6	5.2	-	37.2 (10)	95.9	98.5	39.77	23.91	26.67	3.28	1.92	0.95	9.49

Notes:

(a) Number in brackets represents reaction time in minutes.

5.02 kg/cm^2. After five minutes the rate of increase in pressure as recorded by the transducer was very low and it appeared that a high reaction would not be obtained for a considerable length of time. Pore pressure reactions were therefore discontinued for the remaining tests. Pore pressure reactions for the remoulded specimens were much higher, but one unsatisfactory reaction was obtained for the test at a cell pressure of 5.2 kg/cm^2. Such low reactions for the undisturbed specimens were not expected since the coefficient of permeability was in the order of 10^{-8} cm/sec and initial degrees of saturation were greater than 93 per cent. It appeared that much of the pore water in the undisturbed specimens was bound or adsorbed by the clay particles. Remoulded specimens were at a much higher initial moisture content and even though at about the same degree of saturation as the undisturbed specimens, exhibited much higher reactions indicating that most of the pore water was free.

Table 5.4 also shows that there was a considerable discrepancy between the consolidation moisture content (obtained from the initial moisture content less the volume of water pushed out of the specimens during consolidation) and the final moisture contents. For the undisturbed specimens, the final moisture contents were larger than the initial moisture contents for all but one test. In all the tests, burette readings appeared to be too high and in order to indicate final moisture contents greater than initial, they

would have to record swell or water intake which was not observed during consolidation. It certainly would be expected that an increase in moisture content would result from the application of back pressure after the consolidation phase but back pressure was not used on the remoulded specimens. However, faulty back pressure procedure should not be discounted as a cause of the non-conformity in volume changes for the undisturbed specimens since Lowe and Johnson (1960) noted that volume changes would occur if back pressures were applied too quickly. For the tests on Little Smoky Clay Till, back pressures were built up over a period of 10 minutes which may have been much too fast to allow water to move through the pores to dissolve and compress pockets of air.

Other possible reasons for the discrepancies in moisture contents were:

a. Membrane leakage. Hollingshead (1965) has given this cause for similar anomalies found in tests conducted on Bear Paw Shale,

b. Absorbtion of moisture during mounting and dismantling. This was definitely possible, especially for a soil that appeared to have a strong affinity for water (note swell on e log P curves at Appendix F). During both the mounting and dismantling process, the specimen could have imbibed water from the previously saturated porous stones and filter strips.

The curves for deviator stress and $\bar{A}$ versus per cent strain for the undisturbed specimens are illustrated in Figure 5.6. There was not much change in the failure stresses for the tests at cell pressures from 2.2 kg/cm^2 to 1.18 kg/cm^2 indicating that these specimens were acting overconsolidated. This observation was in agreement with the direct shear test result at a normal stress of 0.96 kg/cm^2 in which the specimen swelled during shear.

The pore pressure parameter $\bar{A}$ generally decreased with strain, but at failure the values of $\bar{A}$ appeared to be high for a till which must have been subjected to heavy ice loads in its geological history. Using Skempton's relationship between $\bar{A}$ and overconsolidation ratio, the overconsolidation ratio for Little Smoky Clay Till varied from about three to four. Pore pressures for all tests increased with strain and reached a maximum positive value sometime prior to the occurrence of the maximum principal stress ratio. The pore pressures then decreased with further strain. Only two tests at cell pressures of 1.4 kg/cm^2 and 1.18 kg/cm showed negative pore pressures at failure. From these observations, it was apparent that Little Smoky Clay Till could be considered as being lightly overconsolidated.

The tests at cell pressures of 2.2. kg/cm^2 or less, tended to fail in a brittle manner creating a comparatively severe break in the stress-strain curves. For one of the tests at 1.4 kg/cm^2, this break was accompanied

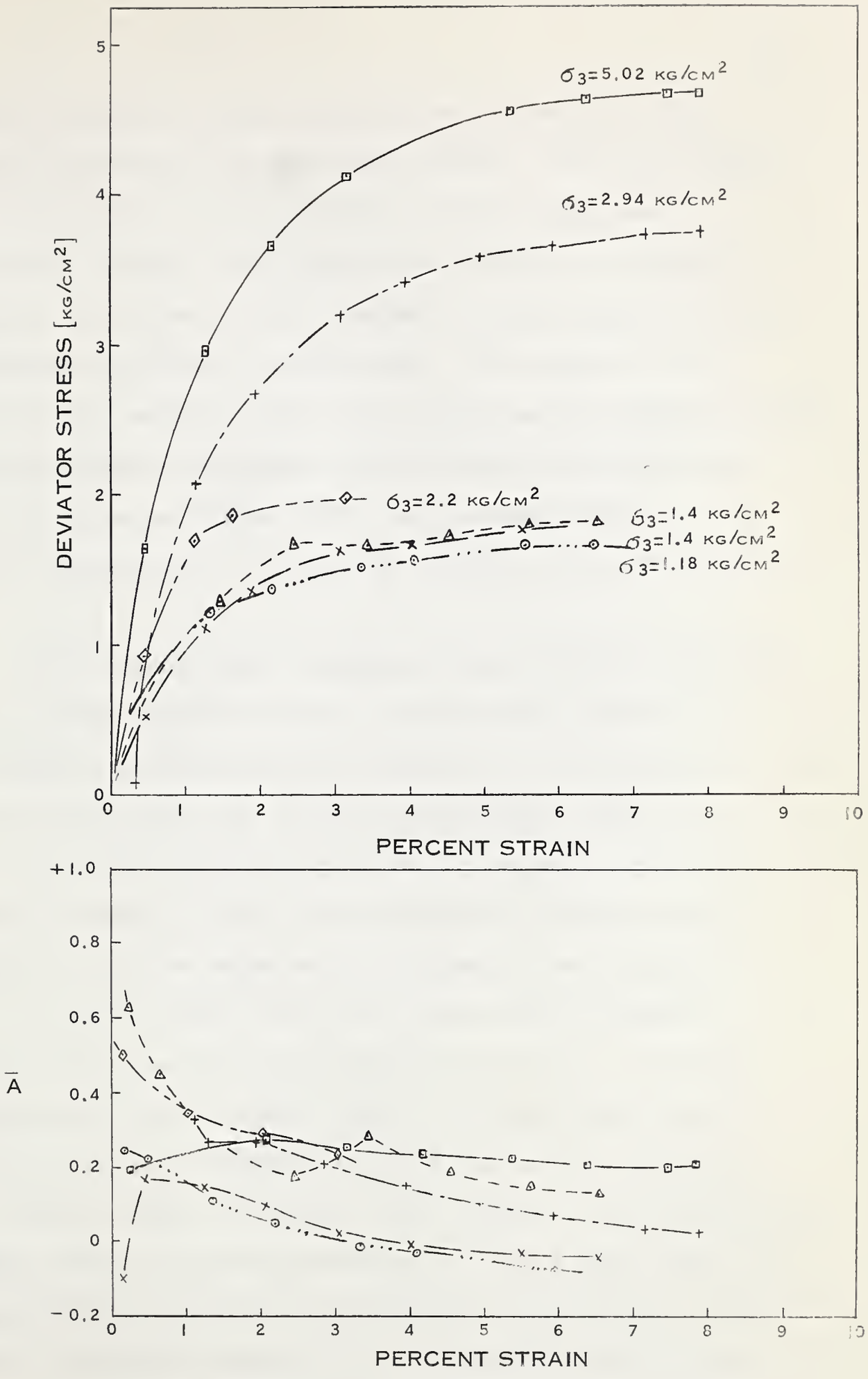


FIGURE 5.6 DEVIATOR STRESS AND Ā VS. PERCENT STRAIN FOR UNDISTURBED SPECIMENS

by a rapid increase in pore pressure as reflected by the relationship between $\bar{A}$ and per cent strain. After the break in the stress-strain curve occurred, stress continued to increase with further strain and pore pressures decreased. The break in the stress-strain curves occurred approximately at the maximum principal stress ratio. The nature of the failure for the tests at lower cell pressures was characteristic of overconsolidated specimens and for comparative purposes it should also be mentioned that the specimen tested at the higher cell pressure of 5.02 kg/cm^2 tended to fail by plastic flow.

The stress-strain characteristics for the remoulded specimens (consolidated from the liquid limit) and for $\bar{A}$ versus strain are illustrated in Figure 5.7. The author could not explain the dip in the stress-strain curve for the test at a cell pressure equal to 1.4 kg/cm^2. For the remoulded specimens, $\bar{A}$ and pore pressures at failure were very high as would be expected for normally consolidated clays. It will be shown later that the effective angle of internal friction for the remoulded specimens was somewhat higher than the residual angle. Although the theory outlined in Chapter 2 suggested that these angles should be similar (Bishop, Webb and Lewin, 1965, found a close similarity for London Clay at medium pressures), it was likely that the prestress effect as outlined by Casagrande and Wilson (1953) caused the remoulded samples to act overconsolidated giving

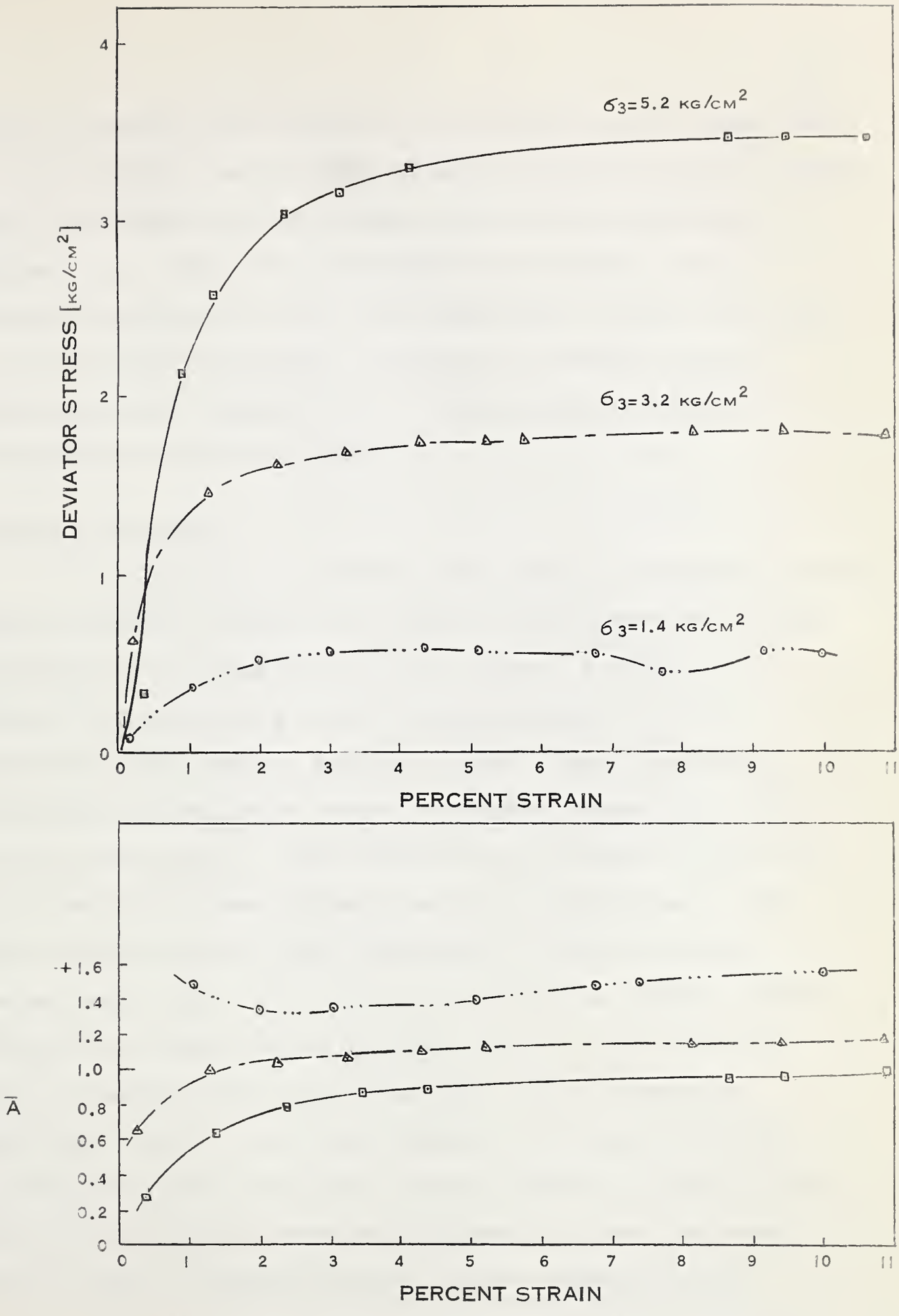
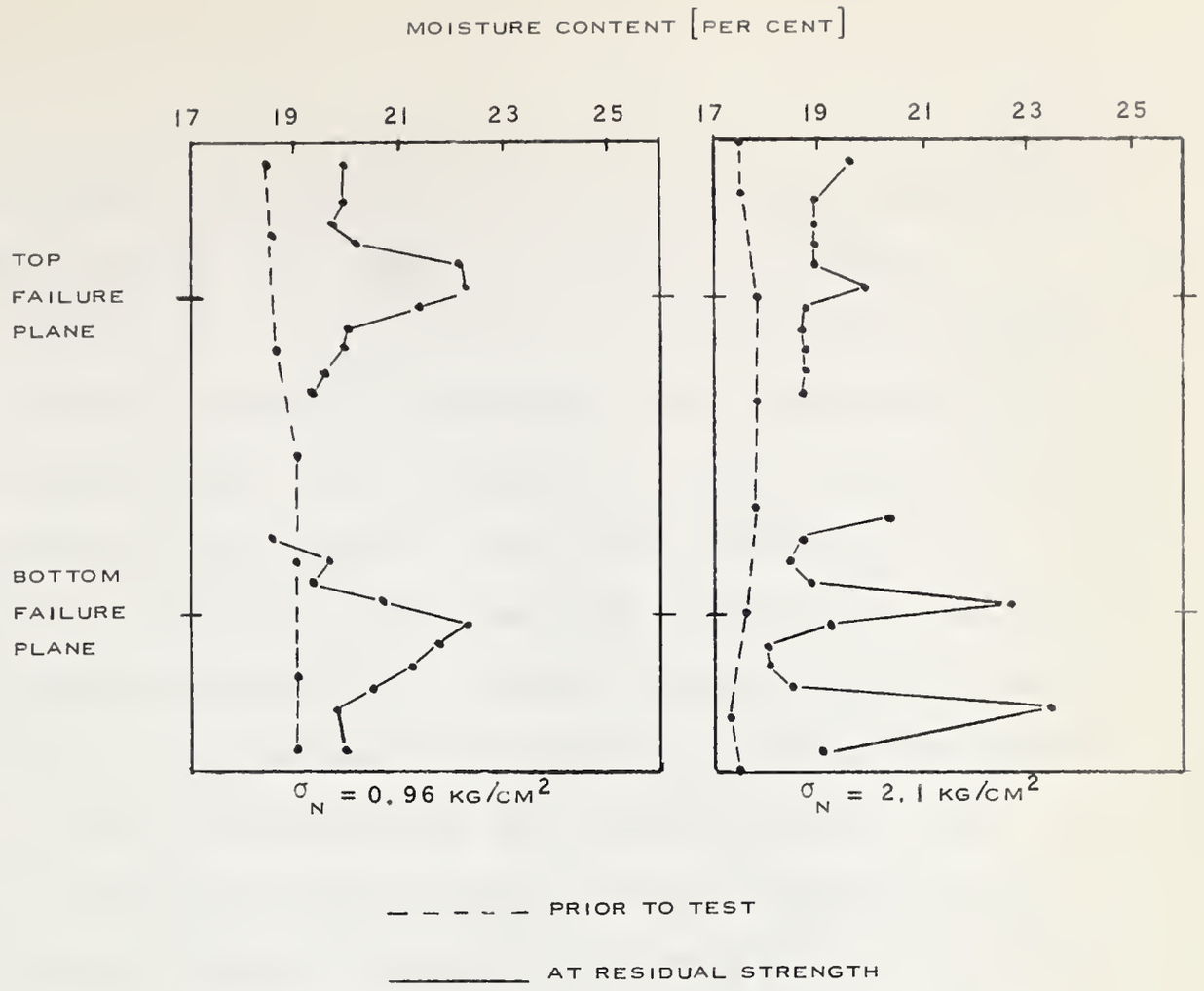


FIGURE 5.7 DEVIATOR STRESS AND Ā VS. PERCENT STRAIN FOR REMOULDED SPECIMENS

higher strengths than obtained from drained direct shear tests. In other words, the effective stresses decreased greatly during shear as exemplified by the high positive pore pressures. Bishop et al (1965) found that the Mohr envelope for remoulded specimens of London Clay exhibited a severe curvature in the low pressure range, the angle of internal friction initially was 21 degrees but at higher stresses tended to be somewhat less than 16 degrees.

Moisture Migration

Figure 5.8 illustrates the results of moisture contents taken vertically through the centre of the direct shear test specimens at the completion of the tests. A total of 22 moisture contents were taken at approximately 0.1 inch intervals away from the failure planes. The illustrations indicated that moisture tended to migrate toward the failure planes during shear. This migration was severe for the test conducted at a normal pressure equal to 0.96 kg/cm^2. This test was the only one that exhibited a volume increase during shear (Fig. 5.5). For the test at the highest normal pressure the migration of moisture was not severe and for the top failure plane there was very little increase in moisture content. There was generally an overall increase in moisture content over the initial condition at the failure plane in all tests. From the diagrams the total thickness of the zones of higher moisture content appeared to be



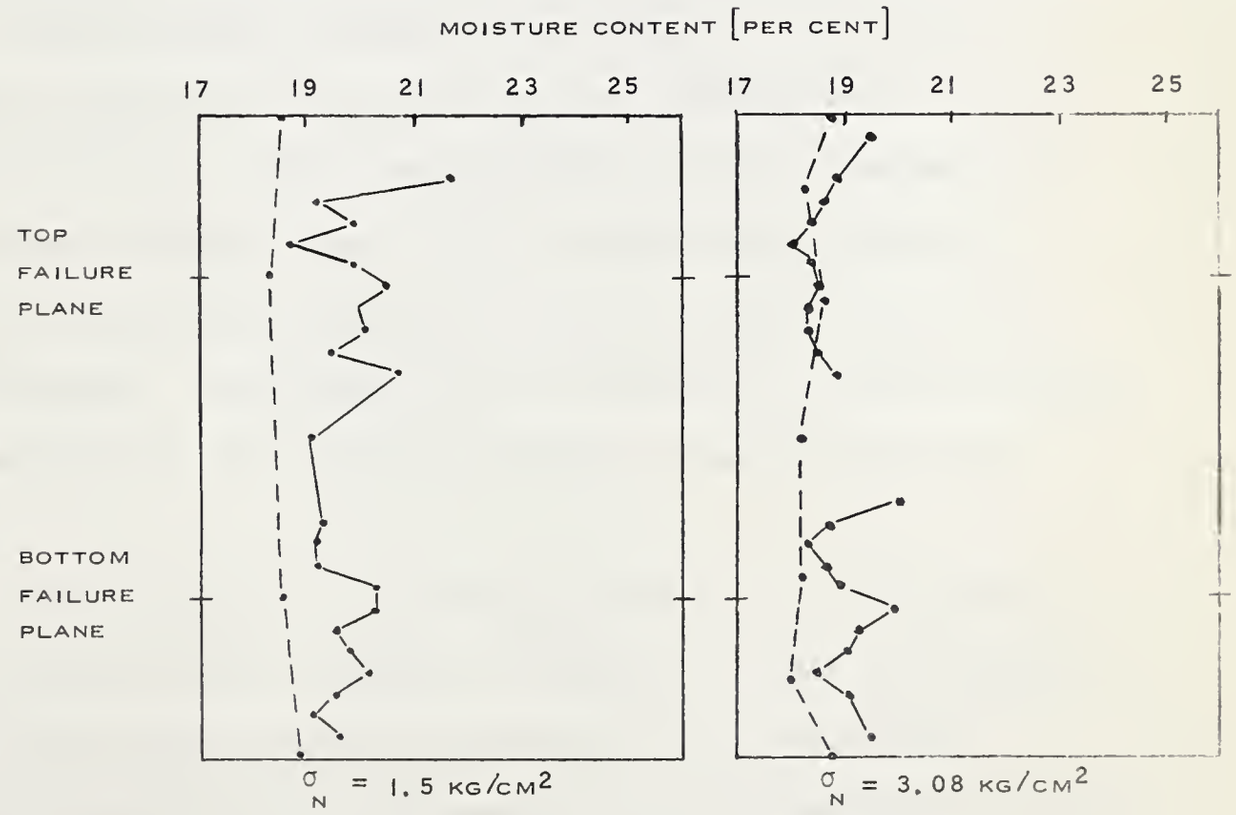


FIGURE 5.8 VERTICAL MOISTURE CONTENT PROFILES FOR DIRECT SHEAR TESTS ON LITTLE SMOKY CLAY TILL

about 0.5 inches. By probing with a spatula, the author found a thickness of very soft zones to be about 0.2 inches.

Figure 5.9 illustrates the moisture content profiles along the failure planes of the direct shear specimens. Five moisture contents were taken along each failure plane at locations shown in the Figure. The centre section of the shear box was displaced toward the right side of the Figure when the test was dismantled. Initial moisture contents were obtained from the trimmings along horizontal planes estimated to be at the same elevations as the failure planes. The increase in moisture content during the test appeared to be greatest for the smallest pressure and smallest for the highest normal pressure. Although the data was not conclusive, the moisture contents were comparatively higher at the ends of the failure planes on the right of the specimens or in the direction of shear. This was probably due to a decrease in normal stress in this area. All the profiles certainly suggested non-uniform shearing stresses and/or non-uniform normal stresses. The shear forces obtained in the tests were then a measure of the average strength along the failure planes.

The increase in moisture content at the failure plane in the vertical profiles of Figure 5.8 was the same as that for the centre moisture content of the horizontal profiles in Figure 5.9. For example, in the vertical profile, the top failure plane for the test at a normal stress equal

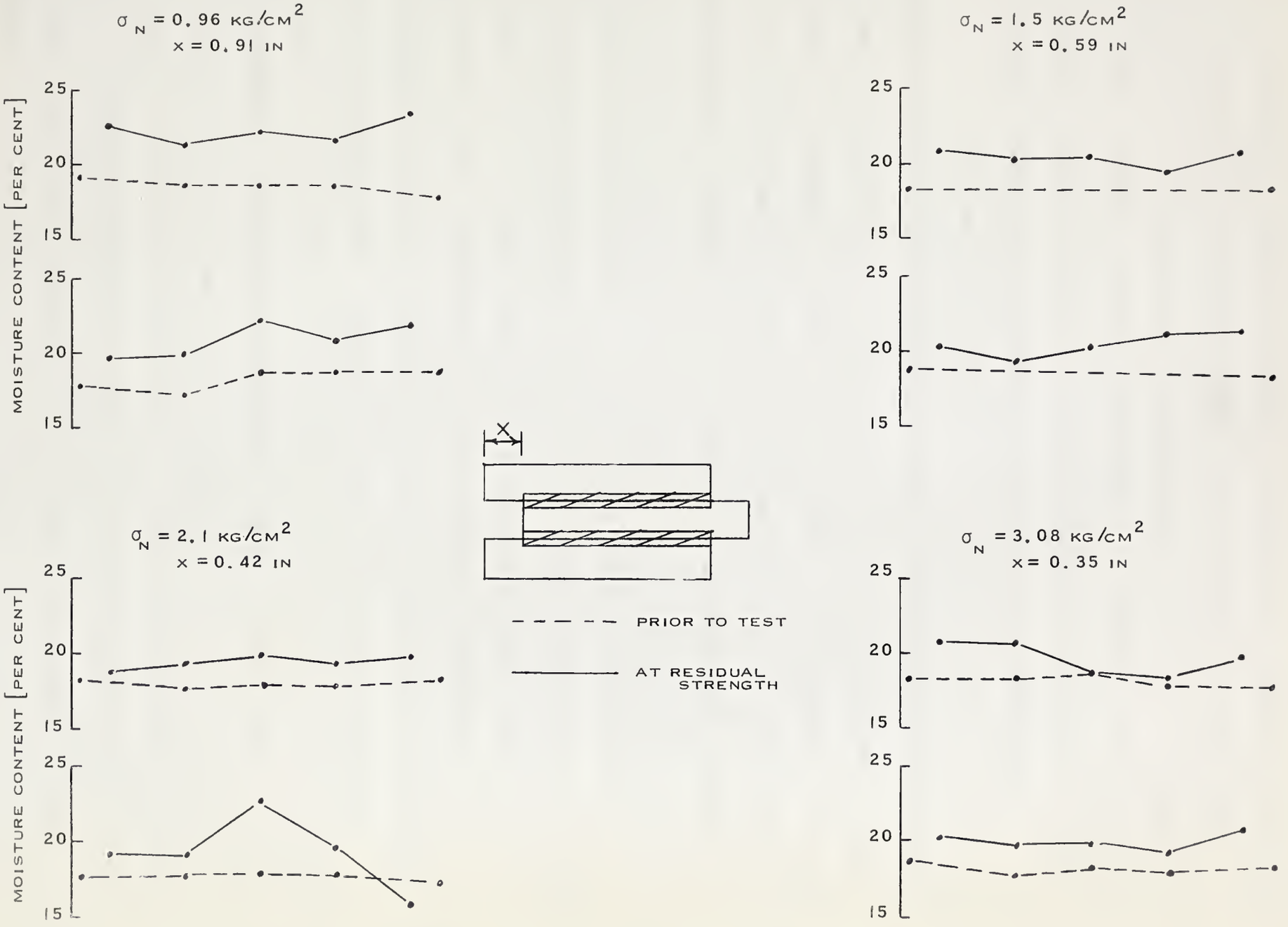


FIGURE 5.9 MOISTURE CONTENT PROFILES ALONG FAILURE PLANES FOR DIRECT SHEAR TESTS ON LITTLE SMOKY CLAY TILL

to 0.96 kg/cm^2 had an increase in moisture content of 3.6 per cent and this was equivalent to the increase in moisture content in the centre of the horizontal profile. Although only one vertical profile was taken, it appeared that similar profiles would be found if taken vertically anywhere in the specimen indicating a migration of moisture throughout.

The increase in moisture content at the failure planes varied from zero for the top failure plane at a normal stress of 3.08 kg/cm^2 to 5.1 per cent for the bottom failure plane at a normal stress of 2.1 kg/cm^2. Considering the slope of the log strength-moisture content curves for the undisturbed specimens in Figure 5.16 the increase in moisture content of 5.1 per cent represents a decrease in strength equivalent to about two log cycles. It was interesting to speculate if unconfined compression tests on specimens soaked to a moisture content of 23.4 per cent (18.3 per cent natural moisture content plus 5.1 per cent) would yield sufficiently low strengths to give a factor of safety of unity for the slide at the bridge site. Skempton (1964) found that moisture migrated toward the failure planes in situ and that unconfined compression strengths for samples taken in the zone of higher moisture content immediately adjacent to the failure plane yielded a factor of safety of 1.12 for the Jackfield Slide. No unconfined compression tests were conducted at this higher moisture content and so the analysis could not be made.

In order to determine the final moisture contents of the triaxial specimens with reasonable accuracy, three were taken vertically through each specimen. Three moisture contents were certainly not conclusive evidence, but the profiles indicated that moisture migrated toward the centre for the tests on undisturbed specimens but migrated towards the ends for the remoulded specimens (Figure 5.10). In all cases, the moisture content at the centre of the specimen was in or very close to the zone of failure. Olsen (1960) found similar results and made the following comments:

a. "It will not be possible to measure the pore pressure in the failure zone at the base pedestal unless the shear stresses are applied slowly enough to allow this migration to take place. The concept that undrained tests can be carried out quite rapidly if the compressibility of the measuring system is taken into account is therefore fallacious."

b. "In overconsolidated specimens the direction of migration depends upon whether positive or negative pore pressures are set up during shear."

c. "In highly overconsolidated samples the pore pressures at failure are negative and moisture should migrate from the ends of the specimen toward the failure zone."

From the statements made by Olsen, it appeared that the undisturbed Little Smoky Clay Till was overconsolidated and possibly heavily overconsolidated although evidence mentioned previously in this Chapter had indicated a low preconsolidation

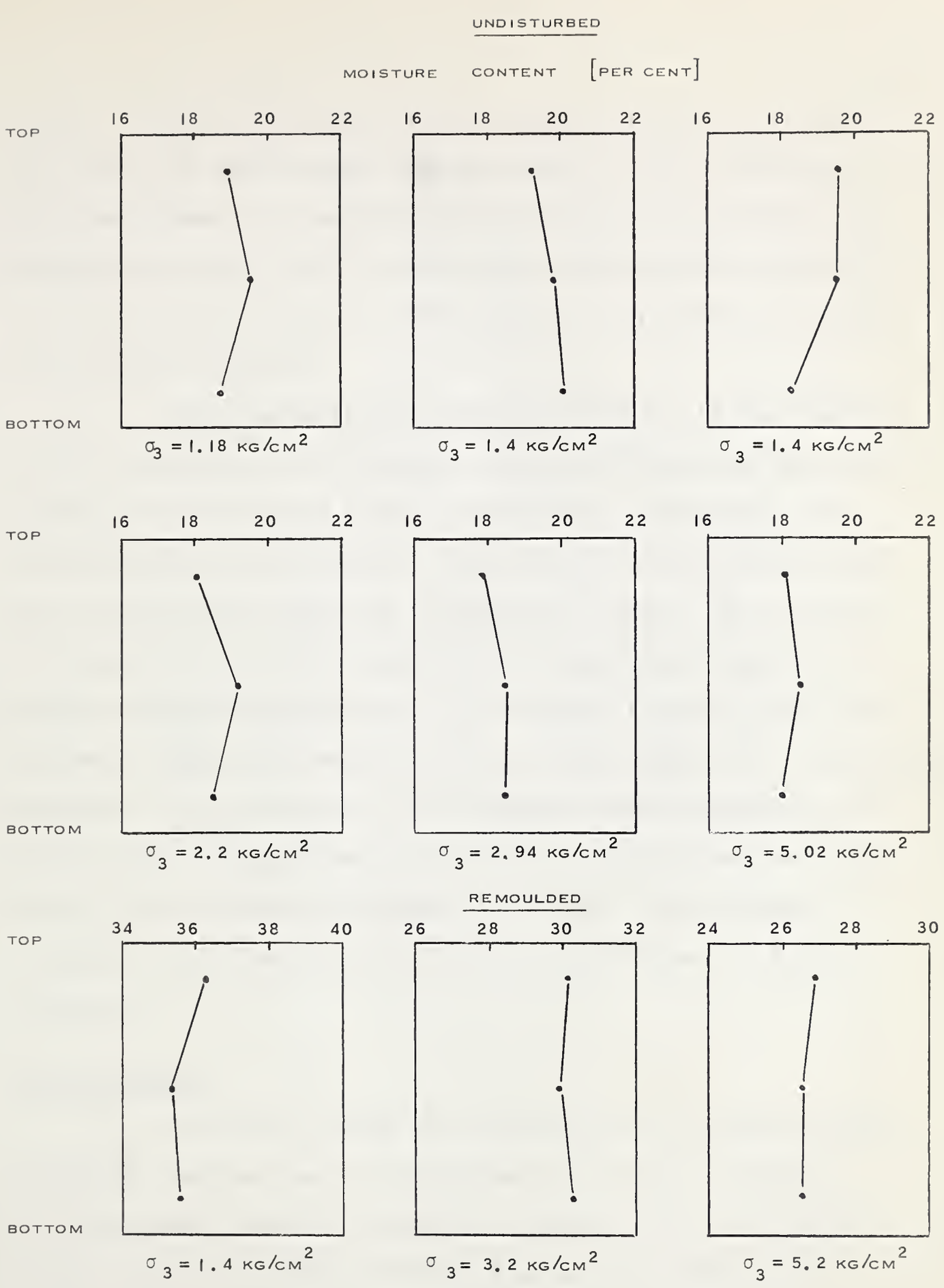


FIGURE 5.10 FINAL MOISTURE CONTENT PROFILES FOR TRIAXIAL UNDRAINED COMPRESSION TESTS ON UNDISTURBED AND REMOULDED SPECIMENS OF LITTLE SMOKY CLAY TILL

load. If the strain rate of 0.11 per cent per hour used for the tests on undisturbed specimens was too fast resulting in the measurement of positive pore pressures at failure, then slower rates may have the effect of shifting Mohr circles in the direction of higher normal stress, decreasing the angle of internal friction.

From the moisture content profiles, it was reasonable to conclude that dilation was occurring along the failure plane for the drained direct shear tests. There was also some evidence that dilation took place in the failure zones for the triaxial undrained compression tests. Yet in only one test did the total volume of the specimen increase in direct shear (normal stress = 0.96 kg/cm^2) and in only two triaxial undrained compression tests were negative pore pressures measured. The author can only suggest that deformation in thin zones accompanied by dilation can take place while the rest of the specimen decreases in volume. This anomaly requires investigating and should be considered in future research.

Mohr Diagrams

The Mohr circles and envelopes for the undisturbed triaxial specimens are illustrated in Figure 5.11 along with the Mohr envelope previously obtained for the Little Smoky Clay Till (Hardy, Brooker and Curtis, 1962). The failure criteria used for the 1965 tests was tangency of

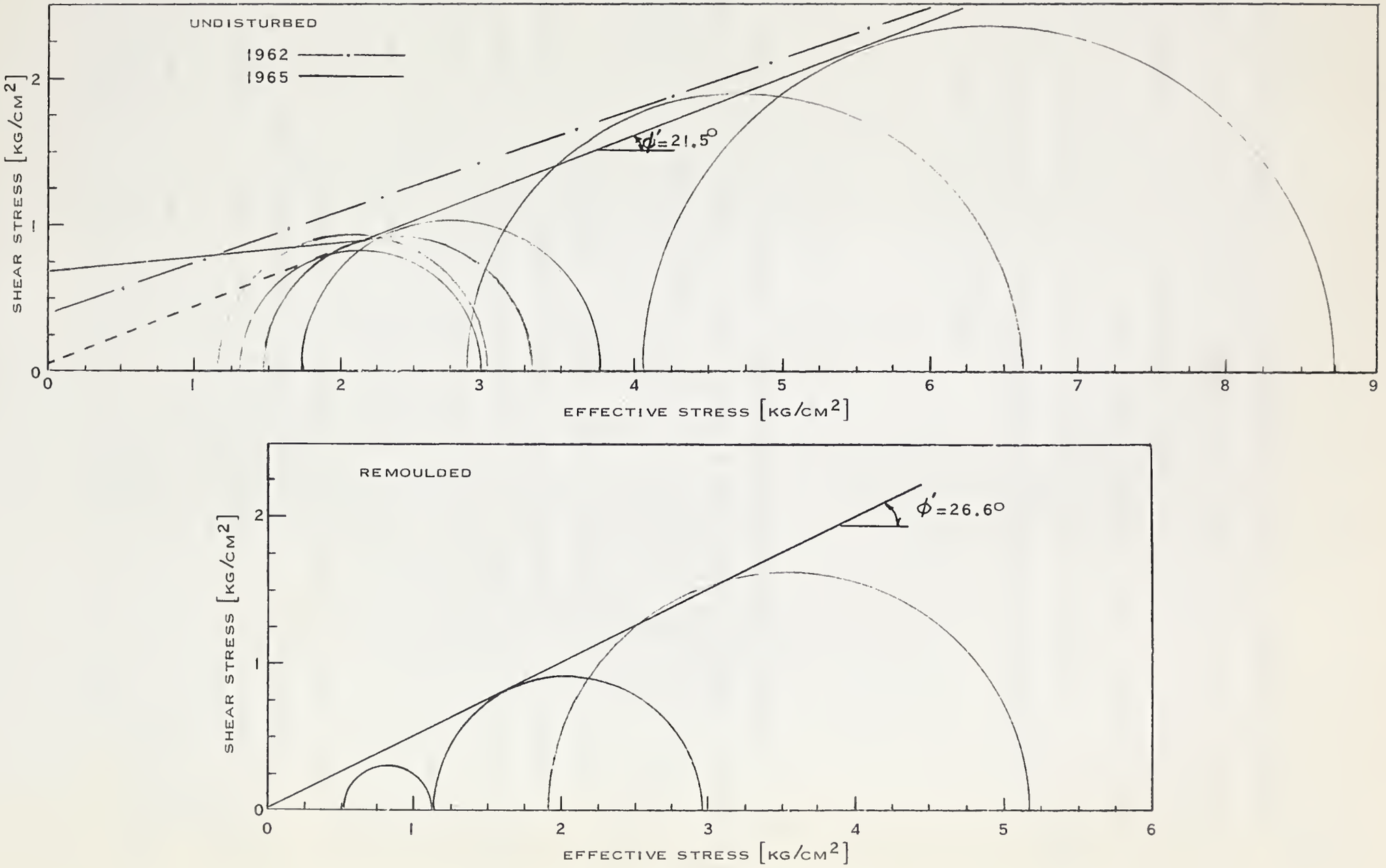


FIGURE 5.11 MOHR DIAGRAMS FOR UNDISTURBED AND REMOULDED LITTLE SMOKY CLAY TILL IN TRIAXIAL COMPRESSION

the Mohr envelope to the maximum deviator stress circles. The use of the maximum principal stress ratio as a failure criterion did not change the position of the Mohr envelope. Due to the fact that strengths changed very little at low cell pressures of from 1.18 kg/cm^2 to 2.2 kg/cm^2, the author considered that a preconsolidation load* of approximately 2.2 kg/cm^2 was indicated.

---

* Ordinarily, the term preconsolidation load refers to the maximum load that the soil has ever been subjected to in its history. For a medium to heavily overconsolidated soil, the value of the preconsolidation load may be in the range of from 20 to 150 kg/cm^2 which represents the weight of glacier ice and/or sediments that covered the soil in the past. However, it is possible that the results of laboratory strength and consolidation tests may show a pseudo-preconsolidation load having a much lower value than would be indicated by the geologic history of the soil. This pseudo-preconsolidation load may be equivalent to the in situ overburden pressure. In this case, it is realized that the use of the term preconsolidation load as applied to the lower value (2.2 kg/cm^2 for Little Smoky Clay-Till) is incorrect. Evidence is presented later in support of a preconsolidation load of about 24 kg/cm^2 for Little Smoky Clay-Till. Nevertheless, the term preconsolidation load will be used when referring to the value of 2.2 kg/cm^2 for Little Smoky Clay-Till due to the lack of more suitable terminology.

The saturated unit weight of the Little Smoky Clay-Till was 133 pounds per cubic foot and samples were obtained at a depth of about 40 feet. Since no water table was found during the drilling of borehole 3A, the in situ effective stress at the time of sampling was about 2.7 kg/cm^2. For an in situ effective stress of 2.2 kg/cm^2, the water table would have to be 15 ft above the sample. This condition was certainly possible and it then appeared that the preconsolidation load of 2.2 kg/cm^2 was a reflection of the effective overburden pressure.

From Figure 5.11 it was evident that the 1965 strengths were less than those obtained previously. This could have been the result of the following:

a. Degree of Saturation: Attempts were made to achieve a high degree of saturation in the 1965 test program by application of back pressure. If this was not done in the 1960 tests, then the higher degree of saturation could lower the strengths (Bishop, Alpan, Blight and Donald, 1960).

b. Test Duration: The 1960 tests were conducted over a duration of from 150 to 200 minutes (Brooker, 1965)

whereas the average time to failure for the 1965 tests was 5,000 minutes. The longer duration may have caused a rheological decrease in undrained strength (Bishop and Henkel, 1962) besides facilitating the measurement of the pore pressures in the failure zone at failure which would tend to have a smaller positive value. Whitman (1960) cited results in which the undrained strength of Boston Blue Clay decreased 12 per cent with a change in strain rate from 0.17 per cent per minute to 0.001 per cent per minute. A similar relationship for the Little Smoky Clay would account for much of the strength difference observed in Figure 5.11.

If more tests were conducted at lower cell pressures in 1960, the author was of the opinion that a small preconsolidation load of 2.2 kg/cm^2 would have been obtained also. A readjustment of the 1960 envelope to account for this preconsolidation load would probably have resulted in the overall difference between the 1960 and 1965 results being much reduced.

The Mohr envelope and Mohr circles for the remoulded specimens are also at Figure 5.11. The failure criteria was maximum deviator stress. The best fit line to the three Mohr circles defined an envelope that gave a negative cohesion intercept. The author then obtained the best fit envelope that passed through the origin of the Mohr diagram giving an effective angle of internal friction of 26.6 degrees.

The Mohr envelopes for all the strength tests con-

ducted are shown in Figure 5.12. The strength envelopes for the remoulded and undisturbed triaxial tests are those that were illustrated in Figure 5.11. The points representing the direct shear results were the average of the stresses obtained from full and no area corrections with only the piston friction correction applied to the shear forces measured. The following observations were made from Figure 5.12:

a. The peak strengths from the direct shear tests were above the Mohr envelope representing the undisturbed triaxial test results.

b. Although there was some scatter in the data, the peak strengths from the direct shear test verified the small preconsolidation load of 2.2 kg/cm^2.

c. The best fit line through the residual strengths obtained from the direct shear box defined an envelope that closely agreed with that portion of the Mohr envelope for the undisturbed triaxial tests that represented normal consolidation. The residual angle of internal friction, ϕ'_r was 22.6 degrees and c' was very close to zero.

d. The angle of internal friction representing the remoulded strength tests was 26.6 degrees and was therefore considerably higher than the residual angle ϕ'_r. The remoulded strengths were also greater than the undisturbed strengths defined by the triaxial tests.

In the opinion of the author, the strain rate used

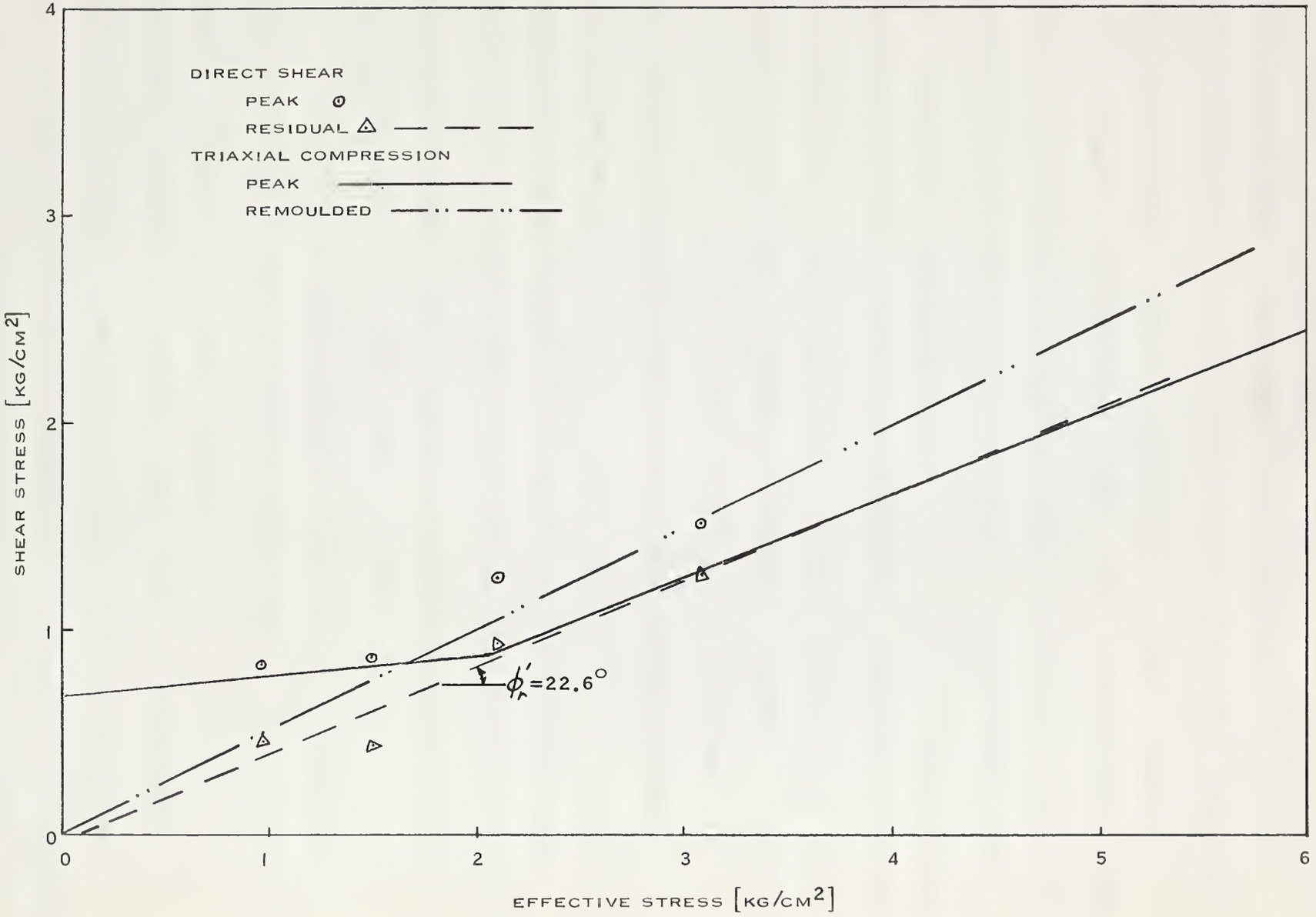


FIGURE 5.12 COMPARISON OF PEAK, RESIDUAL AND REMOULDED STRENGTH ENVELOPES FOR LITTLE SMOKY CLAY TILL

in the direct shear tests was too high. This fact combined with extra strength imparted to the specimens by surface energy would probably account for the higher peak strengths of the direct shear tests as compared to those obtained from the triaxial compression tests. Earlier in this chapter, reasons were given for suspecting that the strain rates for the direct shear tests were too high. For specimens that exhibit negative pore pressures in the failure zone during drained shear tests, which appeared to be the case for the Little Smoky Clay Till, too rapid a strain rate would have resulted in too high a strength (Hvorslev, 1960). A correction for surface energy could not be applied to the direct shear specimens since only one test exhibited an overall volume increase during shear (normal stress = 0.96 kg/cm^2). The moisture content profiles indicated that dilation was occurring and the work done due to expansion in the failure zone would no doubt have resulted in additional shearing resistance (Hvorslev, 1960; Wroth, 1958).

There was good agreement between the residual strength and the normally consolidated portion of the peak strength envelope. At test pressures less than the preconsolidation load of 2.2 kg/cm^2, there was a considerable drop from the peak to residual strength which conformed with the theory outlined in Chapter 2. It would be reasonable to expect the residual strengths to be representative, since drainage must have been fairly complete as indicated by the

volume changes in Figure 5.3, 5.4 and 5.5, which were comparatively small at the ultimate strength portion of the stress-deflection curves.

It was previously mentioned that the high positive pore pressures developed by the remoulded specimens in shear would cause a "prestress" and higher strengths. The results confirmed this. The author initially thought that the testing of remoulded specimens (normally consolidated from a slurry) would provide a convenient method of determining the residual angle of internal friction rather than conducting direct shear tests out to large strains. Unless a more suitable procedure for producing remoulded specimens is obtained, this method is unsatisfactory since evidence has shown that the difference between the remoulded and residual angles of internal friction in the low pressure range is large. The remoulded friction angle was 4 degrees higher than the residual angle and for London Clay, the remoulded angle was 4.9 degrees higher (Bishop et al, 1965).

An estimate of the measurement errors involved in the determination of the residual, remoulded and peak angles of internal friction are summarized in Table 5.5. In Table 5.6, the effect of the errors in Table 5.5 is illustrated by the range of internal friction obtained. The errors were the worst possible combination of those given in Table 4.1 for the direct shear box, except for the piston friction correction, which had already been applied. The variation in the friction

angles, due to measurement errors, was insufficient to explain discrepancies between the remoulded and undisturbed peak triaxial results, nor did it account for the high peak direct shear values. The measurement errors may have accounted for the differences observed between the undisturbed peak triaxial and direct shear residual strengths. The measurement errors for the triaxial tests did not include piston friction or membrane stiffness, but the author felt that piston friction would be low since rotating bushings were used and membrane stiffness would not be sufficiently high to explain the discrepancies mentioned.

Table 5.5

MEASUREMENT ERRORS STRENGTH TESTS

Direct Shear Tests

Stress	Error From True kg/cm^2 Below	Error From True kg/cm^2 Above
Normal Pressure	- 0.013	+ 0.013
Shear Stress	- 0.045	+ 0.037

Triaxial Tests

Deviator Stress	- 0.06	+ 0.06
Pore Pressure	- 0.03	+ 0.03
Cell Pressure	- 0.01	+ 0.01

TABLE 5.6

VARIATION IN ANGLE OF INTERNAL FRICTION
DUE TO MEASUREMENT ERRORS

	Minimum	True	Maximum
Direct Shear Test - Residual	21°35'	22°36'	23°35'
Triaxial Undisturbed	21°14'	21°30'	22°5'
Triaxial Remoulded	25°32'	26°36'	28°18'

The peak strength equation for the Little Smoky Clay Till could be represented by the Krey-Tiedemann equation, $\tau = 2.2 \quad \tan 17.6° + \sigma' \tan 5°$. The residual strength equation was $\tau = \sigma' \tan 22.6°$. Due to the fact that the preconsolidation load of 2.2 kg/cm^2 appeared to be caused by the in situ effective stress, the overconsolidated peak strength envelope could not be considered representative of the in situ strength. In other words, at lesser depths, the envelope representing overconsolidation would be shifted downward on the Mohr diagram and in situ strengths would follow the normally consolidated strength envelope. It may be assumed then that there was no difference between the peak and residual strengths and that the strength equation for both was $\tau = \sigma' \tan 22.6°$.

Residual Strength and Structure

The relationships between ϕ'_r and the per cent finer than two microns as proposed by Skempton (1964) is illustrated in Figure 5.13 with the Little Smoky result plotted. The other points represented case histories presented by Skempton which formed a curving band of values from sands to the ultimate strength of chlorite, talc and biotite minerals. The Little Smoky point plotted slightly high, but generally conformed to the relationship. For information, Bear Paw Shale was also plotted using the angle of internal friction representing the strength for a factor of safety of unity as found by Ringheim (1964). The two Canadian soils followed the trend in Figure 5.13 and the results agreed with the hypothesis that larger percentages of coarser sizes influenced the residual angle of internal friction by preventing to some degree the orientation of clay particles.

In order to investigate further the type of structure immediately adjacent to the failure planes in the direct shear tests, thin sections were made of specimens taken vertically and including the failure planes. The procedure for producing the thin sections is at Appendix C. Each thin section was rotated in plane polarized light under a microscope. If parallel orientation existed, the light coming through the specimen would change in intensity. Figures 5.14 and 5.15 are presentative of thin sections for the direct shear specimens. The photographs were made

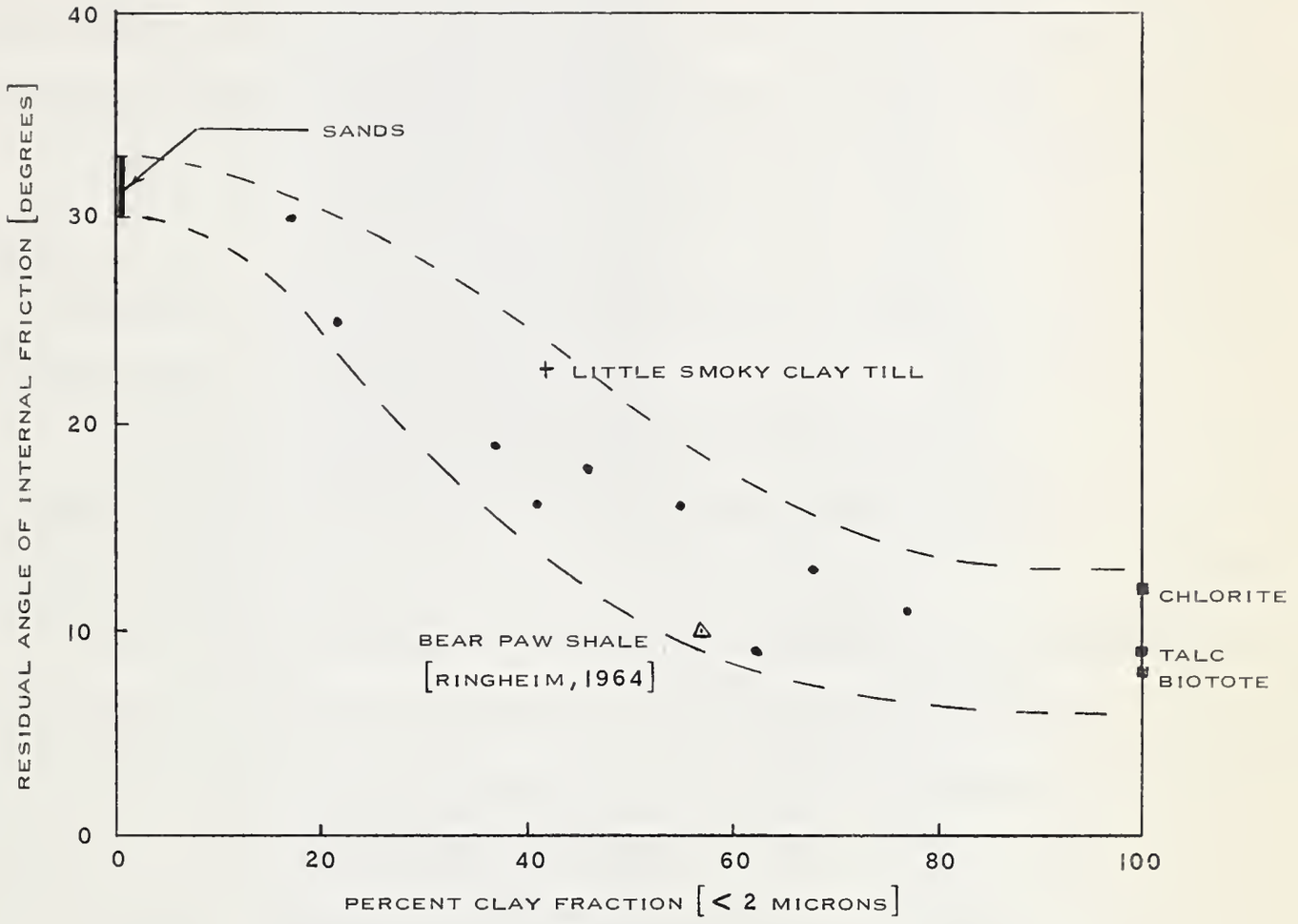


FIGURE 5.13 DECREASE IN ϕ_r' WITH INCREASING CLAY FRACTION [AFTER SKEMPTON, 1964]

through a microscope at a low power of ten but the results were found to be no different at higher powers. As an indication of the magnification, the larger equidimensional grains are coarse silt to medium sand sizes. The two photographs in each figure cover the same field but one was taken with the section rotated 45 degrees with respect to the other.

The following observations were made from the thin sections:

a. Little Smoky Clay Till contained a comparatively high percentage of coarse sizes embedded in a clay matrix.

b. Although it was not too obvious from the low power used in the photographs, the failure planes moved around the coarser grains.

c. The two photographs in Figure 5.14 and 5.15 indicated that the light intensity around the failure planes changed slightly with rotation. In Figure 5.14 rotation darkened an elongated area parallel to but at approximately 0.5 mm away from the failure plane. In one of the photos in Figure 5.15, a darkened band about the failure plane could be observed and it had a thickness of about 1 mm.

The reorientation of the clay particles adjacent to the failure plane was very slight (Lerbekmo, 1965). Reorientation was prevented by the coarser sizes but it could also be stated that the effect of the coarser sizes in creating a non-linear failure plane would increase strength. The thin sections supported the relationship between ϕ'_r and the

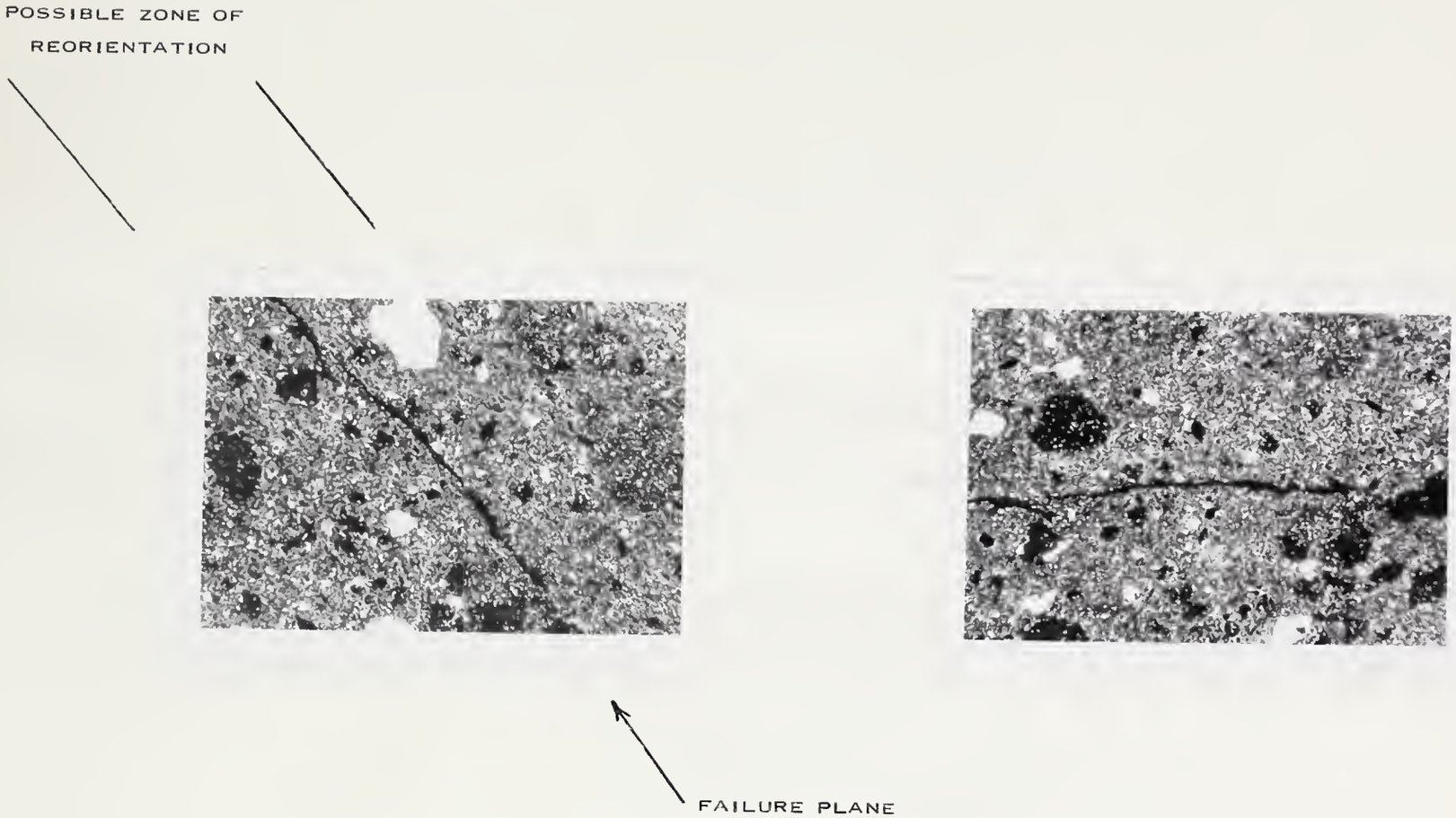


FIGURE 5.14 THIN SECTION OF DIRECT SHEAR SPECIMEN OF LITTLE SMOKY CLAY TILL ROTATED UNDER PLANE POLARIZED LIGHT
[MAGNIFICATION = 19]

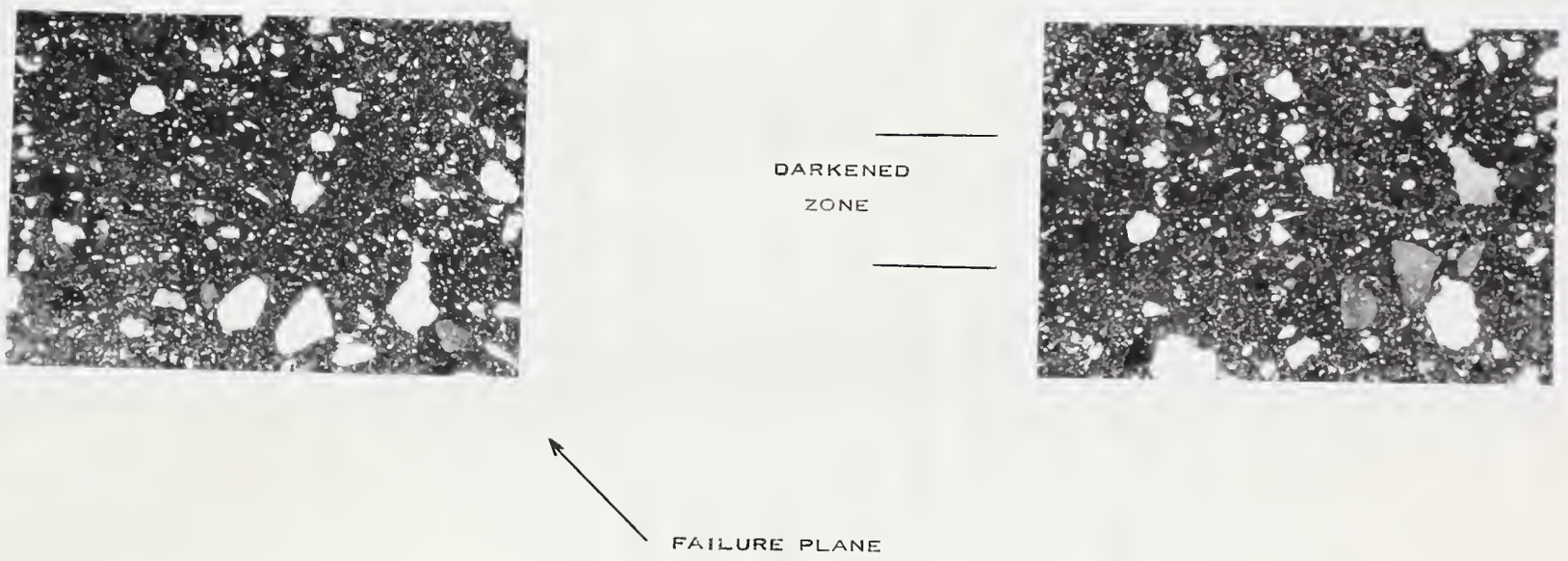


FIGURE 5.15 THIN SECTION OF DIRECT SHEAR SPECIMEN OF LITTLE SMOKY CLAY TILL ROTATED UNDER PLANE POLARIZED LIGHT

[MAGNIFICATION = 19]

percentage of clay sizes and it may also be concluded that tills may always exhibit comparatively high residual angles of internal friction since they contain large percentages of coarse sizes.

The thin sections indicated a very narrow zone of deformation in the order of 1 mm in thickness. This was somewhat smaller than previous estimates (0.2 to 0.5 inches). Nevertheless, the assumption that the full height of the specimen deformed in shear was not entirely valid and the time to failure calculated from the formula developed by Gibson and Henkel (1954) likely overestimated the time required for full drainage to occur. This conclusion did not agree with the results which showed that the peak direct shear strengths were high. The author could not explain this anomaly principally because existing knowledge of narrow deformation zones, dilation and test rates is limited and requires a more detailed investigation.

Moisture Content and Strength

In Figure 5.16 strengths are plotted in the form of maximum deviator stress versus final moisture content for the triaxial tests, residual strength versus final moisture content, and peak direct shear strengths versus consolidation moisture content. The direct shear peak strength line was situated to the left and below that defined by the triaxial tests since the direct shear values were approximately one-

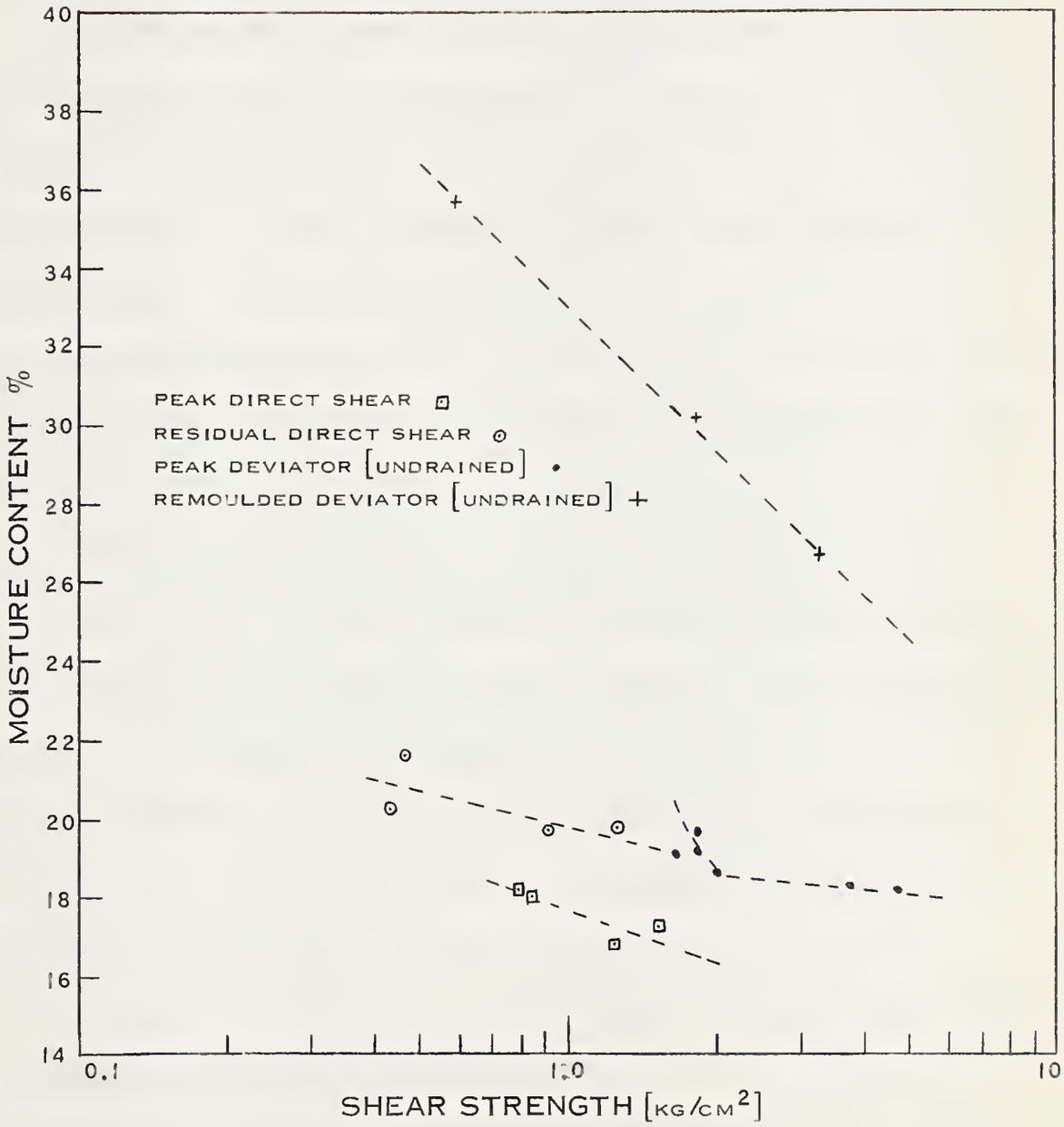


FIGURE 5.16. MOISTURE CONTENT VS. LOG STRENGTH FOR LITTLE SMOKY CLAY TILL

half of the deviator stress and the moisture contents at failure were probably higher than the consolidation moisture content if some moisture migration took place prior to failure.

The strength lines for the undisturbed strengths appeared to be parallel to each other. This was not expected for the direct shear residual and the peak triaxial compression lines, unless it was assumed that both had similar structures at failure. This may have been the case, since heavy particle reorientation was not evident at the residual strength and the orientation existing at failure was probably not much different for both types of test.

Some scatter in points occurred at a moisture content of 18.5 per cent for the undisturbed triaxial tests and the strength line tended to curve upward. This apparently represented the overconsolidation characteristics observed at effective stresses less than 2.2 kg/cm^2. For moisture contents less than 18.5 per cent, the strength changed by 3.2 kg/cm^2 for a moisture content change of 0.5 per cent, whereas for moisture contents higher than 18.5 per cent the strength change was about 0.11 kg/cm^2.

If the strengths at moisture contents less than 18.5 per cent represented the normally consolidated slate, then it would be expected that this portion of the peak triaxial strength line would have been parallel to the strength line for the remoulded specimens (normally consoli-

dated from a slurry). In order to explain this apparent anomaly, it was necessary to investigate the consolidation characteristics of the Little Smoky Clay Till (Figure 5.17). In this Figure, the moisture contents at the end of consolidation were plotted against the logarithm of consolidation pressure for all triaxial tests and one oedometer test. The relative positions of the remoulded and undisturbed triaxial consolidation curves indicated that in the undisturbed state all specimens had been overconsolidated. The intersection of the remoulded and undisturbed consolidation curves gave some indication of the geological prestress which was approximately 24 kg/cm^2 at a moisture content of 13.4 per cent (Bishop, Webb and Lewin, 1965). This value would in all probability not be obtained in an oedometer test since triaxial consolidation with no lateral yield was not undertaken but the indication of a prestress much higher than 2.2 kg/cm^2 was evident. It would also be observed from the undisturbed triaxial consolidation curve, that a sharp curvature occurred at 2.2 kg/cm^2, the value of the lower preconsolidation load.

The stress path plots also exhibited the characteristic curvature of overconsolidation for the undisturbed specimens at all test pressures. The Modified Mohr diagram at Figure 5.18 showed that the stress path plots curved in the direction of higher effective stress. For comparison, the stress path plots for the normally consolidated remoulded specimens (Figure 5.19) curved to the left or in the direction

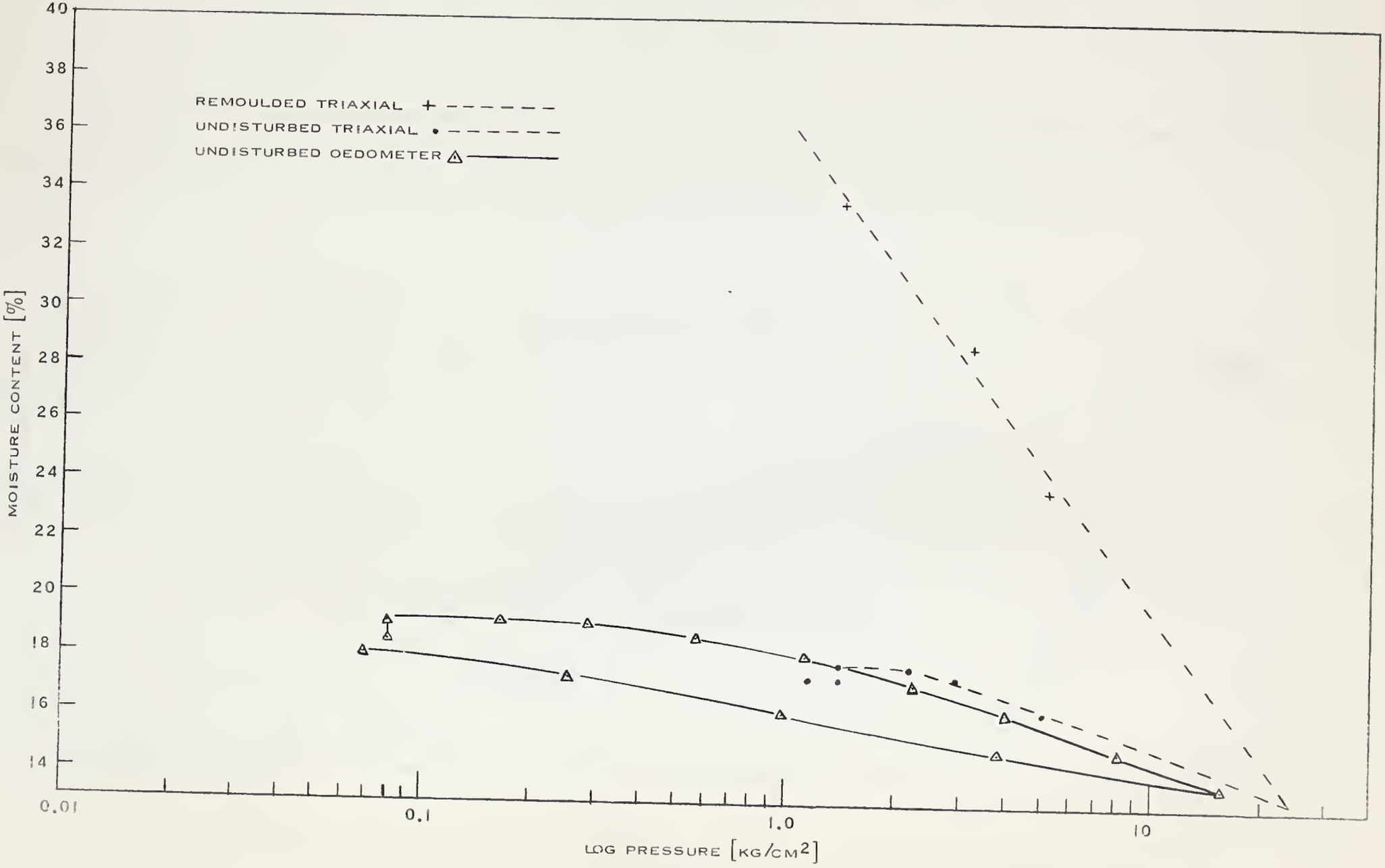


FIGURE 5.17 MOISTURE CONTENT VS. LOG PRESSURE FOR ONE AND THREE DIMENSIONAL CONSOLIDATION OF LITTLE SMOKY CLAY TILL

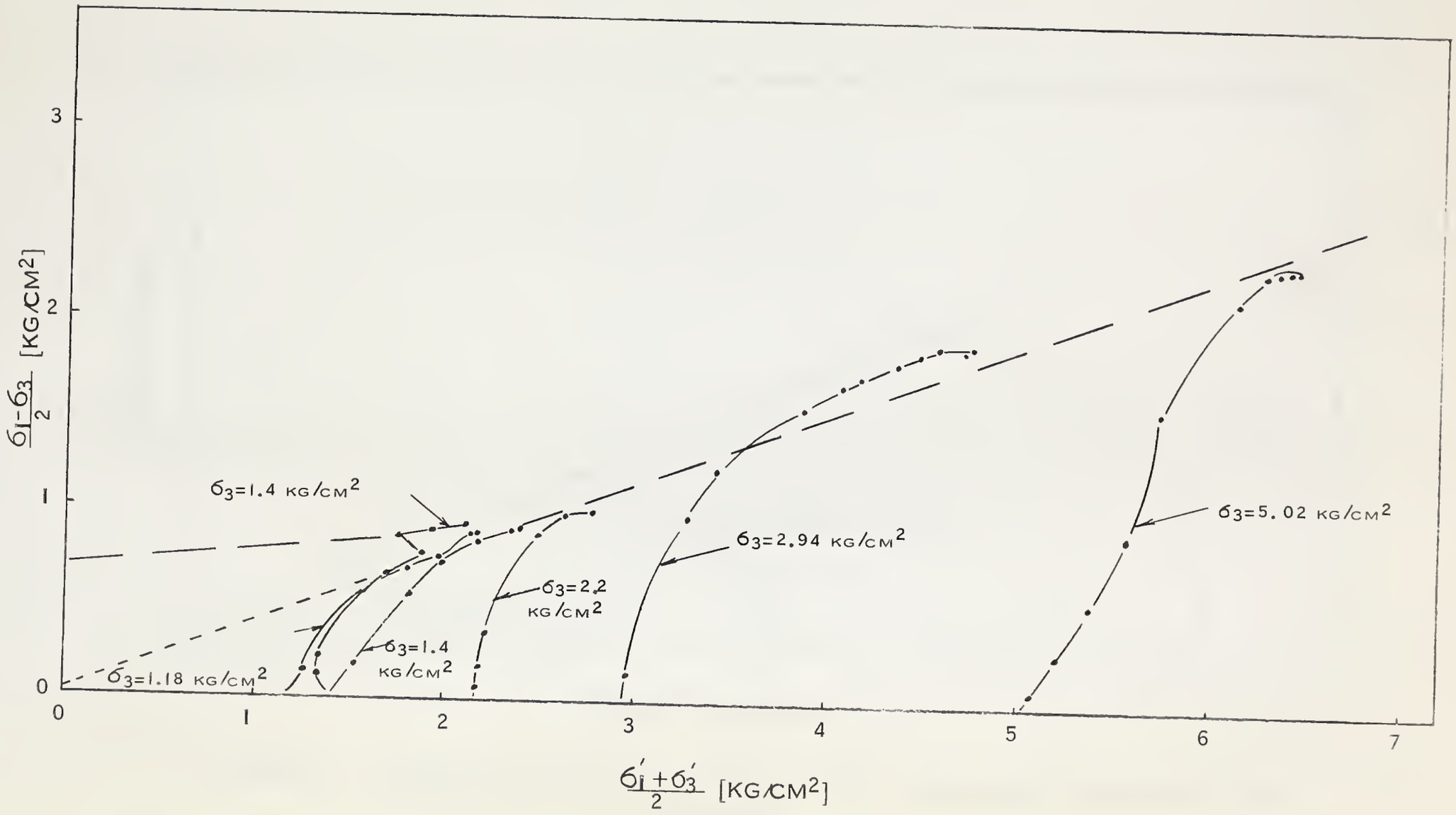


FIGURE 5.18 MODIFIED MOHR DIAGRAM FOR TRIAXIAL UNDRAINED TESTS ON LITTLE SMOKY CLAY TILL UNDISTURBED

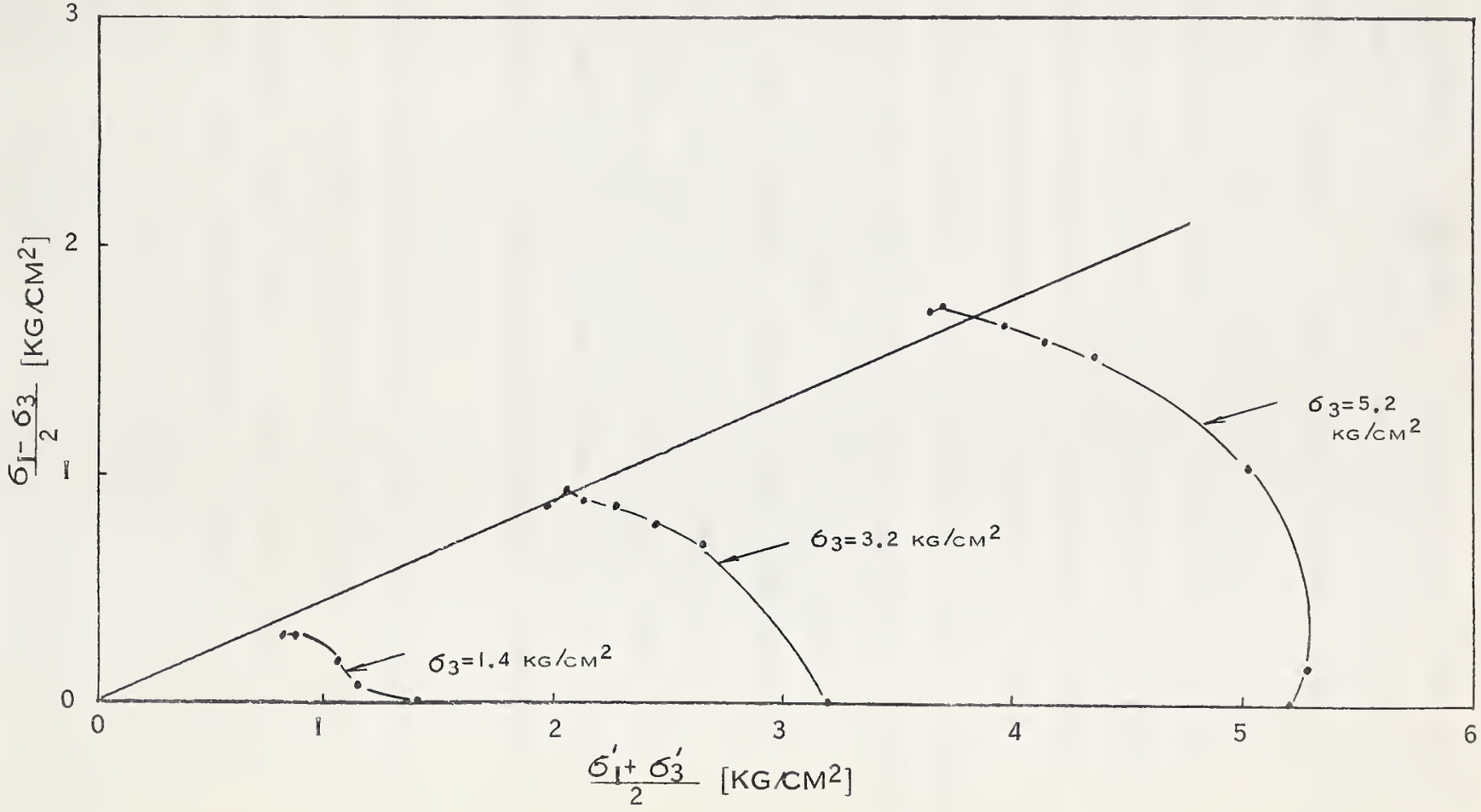


FIGURE 5.19 MODIFIED MOHR DIAGRAM FOR TRIAXIAL UNDRAINED TESTS ON LITTLE SMOKY CLAY TILL REMOULDED

of decreasing effective stress. A Rendulic diagram for the undisturbed specimens at Figure 5.20 also exhibited stress paths that curved in the direction of increasing effective axial and radial stresses. It was then evident that Little Smoky Clay Till had been preconsolidated under a stress considerably higher than 2.2 kg/cm^2 in its geologic history.

It has previously been reported that the supposedly heavily overconsolidated clays of the Edmonton formation exhibited a low preconsolidation load (Painter, 1965). For example, the preconsolidation load determined from strength tests on the Dark Brown Clay Shale was in order of 7 kg/cm^2 (from the Modified Mohr diagram). As a sidelight of Painter's work, this author estimated that the normally consolidated angles of internal friction for Bentonite, Bentonitic Clay Shale and the Dark Brown Clay Shale of the Edmonton formation had values between 17 and 19 degrees which may represent the residual strength for these soil types.

It was possible that Little Smoky Clay Till was never subjected to a high prestress in the past. Henderson (1958) mentioned that 5,000 ft. of ice covered the Middle Till formation but the last readvance in which the Upper Till was deposited was only a "few hundred feet" thick. It was previously indicated that the Little Smoky Clay Till appeared to be the Upper Till formation. A prestress higher than 2.2 kg/cm^2 was then feasible from a geological viewpoint

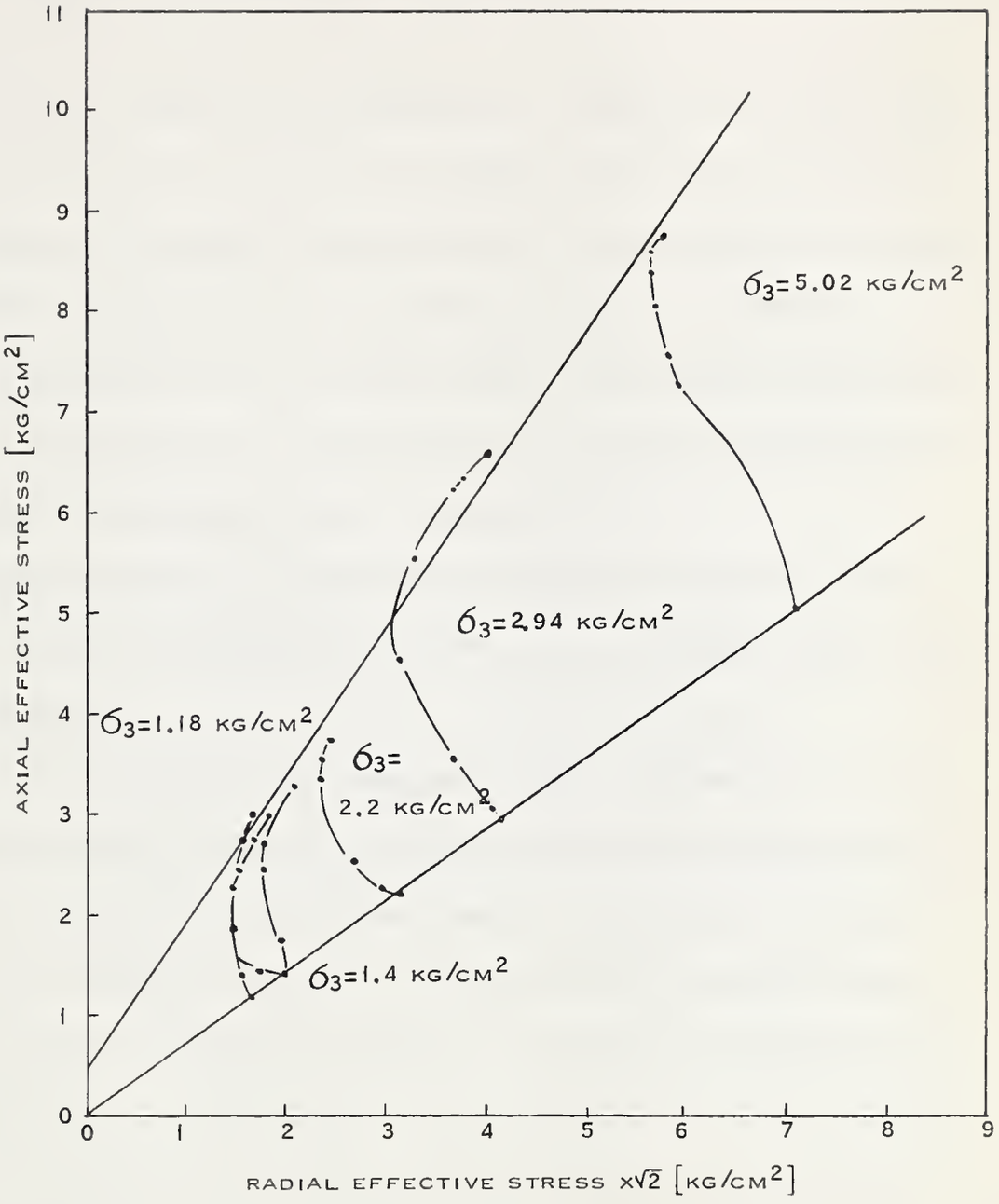


FIGURE 5.20 RENDULIC DIAGRAM FOR TRIAXIAL UNDRAINED COMPRESSION TESTS ON UNDISTURBED LITTLE SMOKY CLAY TILL

and the value of 24 kg/cm^2, although higher than the estimated weight of ice that covered the Upper Till, was not unreasonable.

The literature provided one reason for the disappearance of cohesion in natural slopes over the long term. This was due to the softening action of seepage into cracks and fissures (Terzaghi 1936, Skempton 1964). However, it was noted previously that Little Smoky Clay Till exhibited very little fissuring or jointing and the loss of cohesion by this method appeared to be improbable.

The author attributed the apparent loss of the effects of overconsolidation in the Little Smoky Clay Till to the following:

a. A high degree of rebound after the recession of the glaciers which greatly increased particle spacing and decreased the effect of the intrinsic attractive forces;

b. the Calcium Cation, although not normally associated with high repulsive forces, has given comparatively low strengths on an effective stress basis (Dahlman, 1965);

c. leeching of the free salts during rebound which increased the repulsive forces;

d. the high net negative charge in comparison to particle mass as exhibited by the Montmorillonite clay mineral.

With respect to the type of adsorbed cation, it was interesting to note from the literature that the

potassium cation gave high strength (Dahlman, 1965) and also created a comparatively high degree of fissuring due to its polarizability (Rosenqvist, 1955). Although speculative, this may be the reason for London Clay being fissured but retaining the effects of its preconsolidation load with regard to shear strength. The picture then is one of high strength in intact chunks of the potassium clay but low strength equivalent to the residual value along the softened fissures. The type of cation may then be an important factor with respect to the degree of rebound over the long term, the accompanying degree of strength loss and the development of joints, fissures and cracks. Research on the physical - chemical aspects of consolidation and strength of overconsolidated clays may shed more light on this problem.

Lastly, with respect to the lower preconsolidation load of 2.2 kg/cm^2 (approximately equivalent to the in situ overburden pressure), it was observed that this low prestress had a substantial effect in creating additional strength. The author suggests that the Little Smoky Clay Till had a degree of structure in its undisturbed state. Interparticle bonds may have been formed by carbonate cementation and these provided additional shearing resistance at test pressuresless than 2.2 kg/cm^2. The possibility of interparticle cementation existing in the till was supported by the following experimental evidence:

a. Values of $\bar{A}_f$ greater than zero for tests on

undisturbed specimens at pressures greater than 2.2 kg/cm^2. According to Denissov (1965), overconsolidated clays having values of $\bar{A}_f$ between zero and 1.0 are underconsolidated due to cementation between particles. It could be said that triaxial consolidation destroyed the bonds, setting up conditions for excess positive pore pressures during shear.

b. The comparatively sharp break in the stress-strain curves for tests at pressures less than 2.2 kg/cm^2 (Figure 5.6). For these tests, the bonds were not destroyed during the consolidation phase, but as evidenced by the break in curvature, were destroyed during shear after substantially high strengths were mobilized. It should also be noted that the pore pressure increased abruptly at the break in the stress strain curve for one test (Figure 5.6).

c. The values of $\bar{A}_f$ that exceeded unity for two of the three triaxial tests on remoulded specimens. During the formation of these specimens (consolidation from a moisture content exceeding the liquid limit), interparticle bonds were developed to some degree. In shear, pore pressure exceeded the applied deviator stress, a condition which is a characteristic of an underconsolidated soil (Denissov, 1965). It was surprising that cementation could develop in the short period of time for the formation of the remoulded specimens but there was an indication that such bonding could develop in the Little Smoky Clay Till.

Summary

The soil properties and the strength characteristics of a clay till of medium plasticity were detailed in this chapter. It was found that:

a. The clay in the till was calcium - montmorillonite and that a comparatively high degree of swell and rebound was exhibited by this soil type;

b. the direct shear box method of determining the residual strength was not entirely satisfactory due to the effect of stress reversals and the inability to determine the correct area to calculate stresses.

c. The time to failure to ensure a fully drained test in the direct shear box must be determined by means of a trial and error procedure.

d. In the drained direct shear tests volumes decreased in shear except for one test at a normal stress of 0.96 kg/cm^2. Pore pressures at failure in the undrained triaxial tests were positive except for two tests at cell pressures of 1.18 and 1.4 kg/cm^2.

e. Dilation appeared to have occurred for all tests as indicated by the migration of moisture toward the failure zones. This presented an anomaly since negative pore pressures were not measured at the base of the triaxial specimens except in two tests and volumes decreased for all but one drained direct shear test. There was no explanation for this problem but it was proposed that it was related to the

phenomena of dilation and deformation in narrow zones. Deformation in narrow zones was indicated by thin sections and moisture content profiles taken from direct shear specimens.

f. The peak strength parameters could be expressed by the Krey-Tiedemann modification of the Mohr-Coulomb equation, $\tau = 2.2 \tan 17.6° + \sigma'_f \tan 5°$. The residual strength equation was $\tau = \sigma' \tan 22.6°$. The strength equation for tests on remoulded specimens (normally consolidated from a slurry) was $\tau = \sigma' \tan 26.6°$. It was concluded that the testing of remoulded specimens was not a good method of determining the residual angle of internal friction.

g. The residual angle of internal friction obtained conformed with the relationship of ϕ'_r versus the per cent finer than 0.002 mm as proposed by Skempton (1964). In conjunction with this relationship, it was observed from thin sections that very little reorientation of clay particles occurred along the failure planes at the residual strength which accounted for the comparatively high residual angle of internal friction obtained. It was also observed that there was very little difference in slope for the moisture content - log strength relationships representing the peak and residual strengths, indicating very little difference in structure.

h. The Mohr envelope representing residual strength coincided with the normally consolidated portion of the peak

strength envelope as postulated by Skempton (1964).

i. A low preconsolidation load of 2.2 kg/cm^2 was present and this was about the same value as the in situ effective overburden pressure. Nevertheless, there was additional evidence that the clayey till had been subjected to a much higher prestress in the order of 24 kg/cm^2. The author believed that the explanation of this phenomena could be obtained by a study of the physical-chemical characteristics of overconsolidated cohesive soils. The high strengths at test pressures less than 2.2 kg/cm^2 were likely due to interparticle cementation bonds.

CHAPTER 6

STABILITY ANALYSIS

Description of Site

The west bank of the Little Smoky River at the bridge site is typical of many of the river valleys found in the Peace River district. There is considerable evidence of a great deal of slumping activity occurring in the geologic past which has resulted in the bank being scarred by numerous old slide scarps, at the base of which there is usually ponded surface water. In these ponded areas, marsh grass has grown and it appears that this surficial water remains throughout the year. In the area of the bridge site, the surface was found to be comparatively dry, the only wet area being the creek that runs along a fairly deep cut just south of the bridge (Figure 6.1). There is very little evidence of a developing slide in the vicinity of the pier and the two abutments on the west bank except for a crack in the approach road 150 feet west of abutment 2 and a 4 inch scarp recently observed 50 feet east of pier 1 (Figure 6.2). Both cracks are approximately perpendicular to the bridge centre line and the one on the highway occurs periodically, necessitating repairs to the pavement. The average slope of the bank in the vicinity of the abutments is 12 degrees.

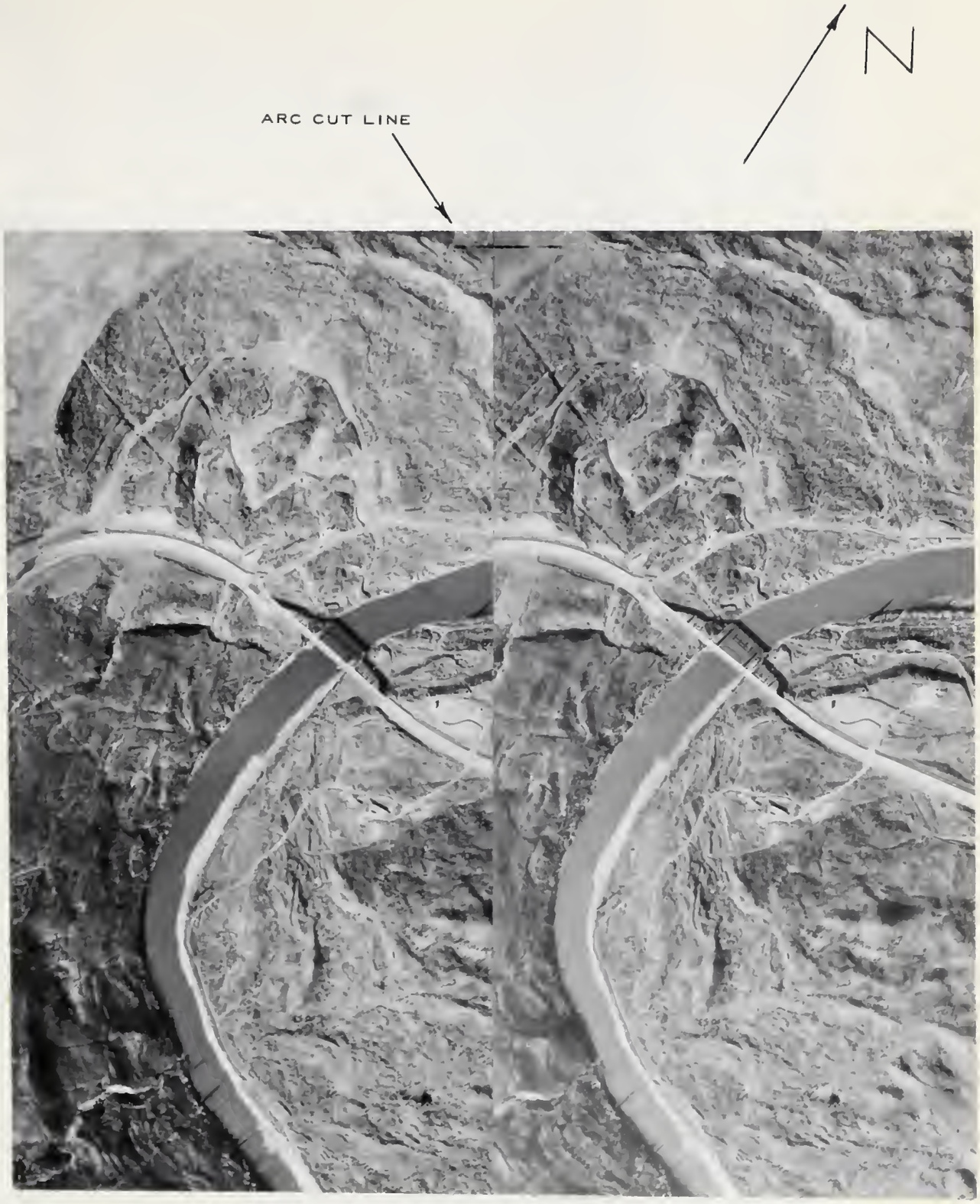


FIGURE 6.1 STEREO PAIR OF LITTLE SMOKY BRIDGE SITE

[PHOTO TAKEN IN 1965, APPROX. SCALE 1 IN.=1000 FT.]

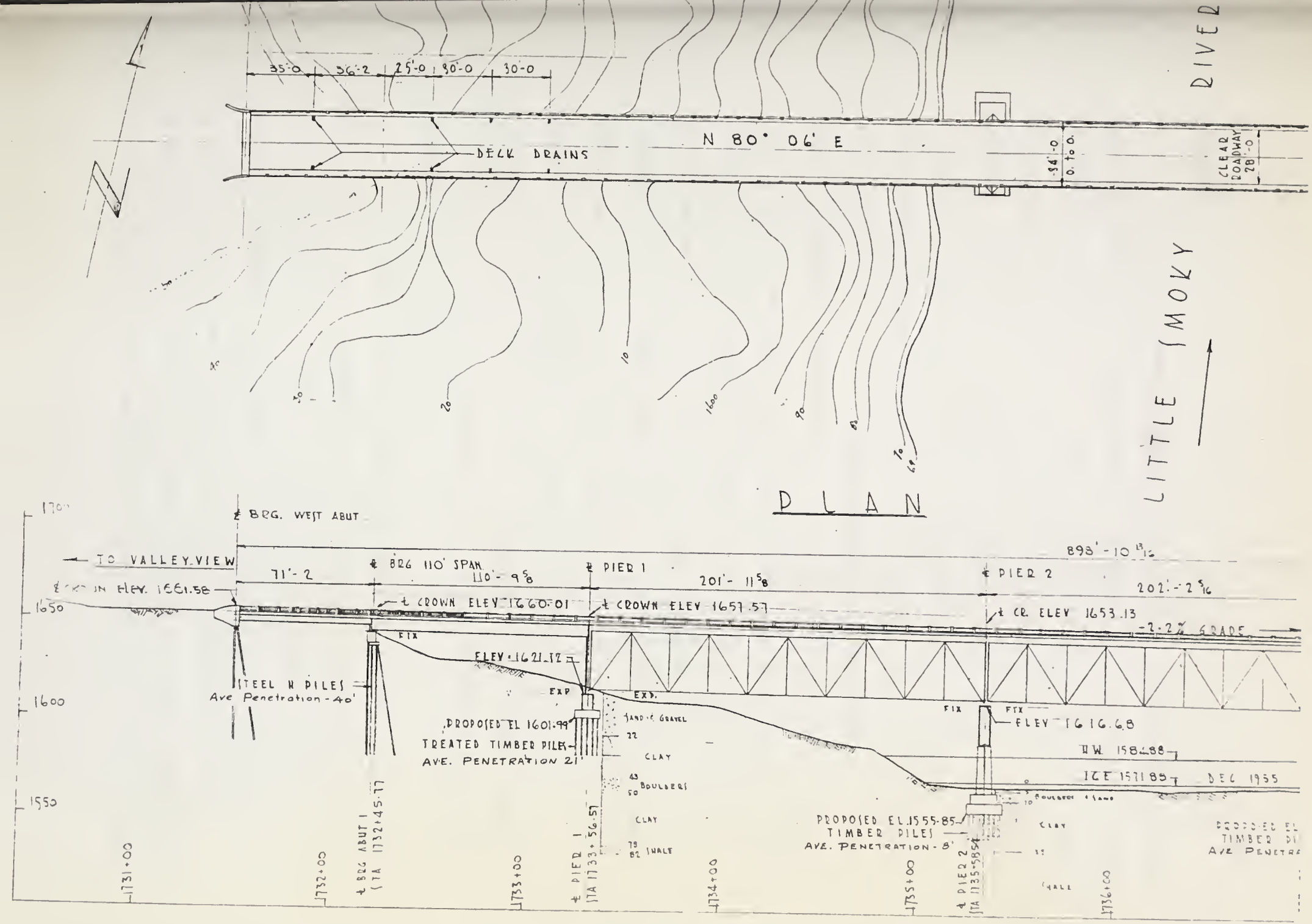


FIGURE 6.2 CROSS-SECTION AND PLAN OF WEST BANK ABUTMENTS, LITTLE SMOKY RIVER

Movement

A cross-section and plan of the bridge on the west bank of the Little Smoky River, 35 miles north of Valleyview, is shown in Figure 6.2. Pier 1 was founded on 20 foot timber piles, whereas abutments 1 and 2 were founded on 40 foot H-piles. The most noticeable movements have been observed in pier 1. Since 1957, pier 1 has moved 50.4 inches in a longitudinal direction towards the river, 22.8 inches vertically downward, and 11 inches upstream in a southerly direction. Corrective action has consisted of extending the concrete cap of the pier toward the river and building it up to correct for the vertical movement. In 1958, the ponds up the slope were drained into the river by trenches and the movements of the pier decreased. In the early part of May 1965, there was an extended period of heavy rainfall. Subsequent to the rainfall, the rate of movement of pier 1 increased considerably and the pier moved 6.5 inches longitudinally from May to September. A graph illustrating this movement is at Figure 6.4.

No definitive records are available for the movement of the two abutments on the west bank, but during the period following the heavy rains in the spring of 1965, abutment 1 moved 0.75 inches in a longitudinal direction. No records have been maintained for the movements of the first pier in the river, although expansion joints throughout the full length of the bridge have narrowed indicating either

movement in this pier, which has fixed connections or movement on the east bank.

Assumptions

The following assumptions were made prior to undertaking a stability analysis:

a. Homogeneous and isotropic soil conditions were assumed to exist throughout the bank.

b. For the circular arc analyses attempted, pore pressures were assumed to be equivalent to the vertical distance from the arcs to the phreatic surface, extrapolated from the maximum elevations of water levels observed in the probe wells. Elevations as observed at the end of October 1960, and about one month later for boreholes 1, 1B, 2 and 3 along the A.R.C. cutline are given in Table 6.1. The location of these boreholes is shown in Figure 6.5. Water levels observed in piezometric installations are also shown in the Table.

c. The Saskatchewan Sands and Gravels were assumed to exist at the site and their elevation was assumed to be at 1550 feet.

d. The peak strength for the Little Smoky Clay Till could be represented by the Krey - Tiedemann equation $\tau = 2.2 \tan 17.6° + \sigma_f' \tan 5°$. The residual strength equation was $\tau = \sigma' \tan 22.6°$. Residual strength was assumed to be applicable for it will be shown later that the movements

now occurring were the result of sliding along old slip surfaces. The large movement along old slip surfaces inferred the mobilization of residual strength.

e. There was no definite evidence that the first pier in the water off the west bank (Figure 6.2) had tilted or moved laterally. The toe of the slide was then assumed to be in the river between this pier and the shoreline.

f. The recent movements were the result of previous sliding activity in the area bounded by the large circular scarp north-west of the bridge (Figure 6.1).

g. The type of failure in the area of the large circular scarp was a series of circular slips that progressed back to the existing scarp.

h. The circular arc type of analysis was therefore applicable.

j. The direction of movement was approximately parallel to the A.R.C. cutline indicated in Figure 6.1.

Discussion

The validity of the assumption concerning homogeneous and isotropic soil conditions could certainly be questioned. Figure 3.1 indicated a rather heterogeneous soil mass as far as soil type was concerned. The agreement between the Atterberg Limits for boreholes 1B, 2 and 3A (Appendix E) was good but the liquid limit for boreholes 1B and 2 appeared to be higher than that for 3A. The natural moisture contents

TABLE 6.1

WATER LEVEL ELEVATIONS IN BOREHOLES
AND PIEZOMETER INSTALLATIONS
LITTLE SMOKY RIVER

Borehole	Depth of Water Levels (ft)	Depth Piezometer Tips (ft)	Max. Piezometer Levels (From Ground Surface) (ft)
1	31 (26 Oct '60) 3 (25 Nov '60) 4.5 (29 Nov '60)	28.5	21 below
1B	21 (1 Nov '60)	21	15 below
2	20 (26 Oct '60) 20 (25 Nov '60) 24.3 (29 Nov '60)	20	12 below
3	50 (27 Oct '60) 9.5 (25 Nov '60) 9.5 (29 Nov '60)	50	2.5 above

Notes:

(1) Location of holes along A.R.C. cutline shown in Figure 6.5

for boreholes 1B and 2 were also higher, especially at depths less than 30 feet. It was mentioned in Chapter 3 that none of the boreholes penetrated the till formation. Therefore, the variation in properties for samples taken throughout the bank would lie in the range of characteristics between the different tills found there. It appeared that the Little Smoky Clay Till was the Upper Till due to the large percentage of clay sizes. This would mean that the samples tested would likely have the lowest residual angle of internal friction of all till types in the bank (Figure 5.13).

In Table 6.1, the Casagrande piezometer readings were higher than the water levels observed in the same borehole except for borehole 1. This was probably due to the fact that the observed water levels in the boreholes did not represent equilibrium conditions due to the short time between readings (one month), or that the piezometer tips were measuring artesian pressures at the depth installed (a wet sand layer was observed at this depth according to Hardy et al, 1962). In any case, the author employed the water levels found in the boreholes to estimate the phreatic line for the stability analyses using circular arcs. The piezometer observations were used for an infinite slope analysis at a depth of 25 feet which closely conformed with the depth of the piezometer tips in boreholes 1, 1B and 2. The assumption that pore pressures on circular arcs were equivalent to the vertical height of water above the arc

was not conservative but this was the best alternative to a flow net.

The location of the Saskatchewan Sands and Gravels was discussed in some detail in Chapter 3. The author estimated that this formation was not far below the greatest depth of boreholes in Figure 3.1, and as a result established their elevation at 1550 feet. This should be verified at a future date by drilling one deep hole in the vicinity of borehole 2.

One important point with regard to the movements occurring in the area bounded by the large circular scarp north-west of the bridge (Figure 6.1) was the amount of material that had been removed. The large circular scarp has a diameter of about 2900 feet (scale 1 in = 1000 ft) and the existing ground surface varied from zero to 300 feet below the top of the valley slopes. At least 10 million cubic yards of material has moved but there was no evidence of its present location in Figure 6.1. The material must have been removed by the river which inferred a comparatively slow movement of sliding material toward the river in the past.

A cross-section of the entire slope taken through the A.R.C. cutline is at Figure 6.5. The dashed line represents the author's estimate of the slope profile before slumping activity had started. From this profile, it was difficult to envisage a large circular slide develop-

ing with a scarp at chainage 20+00 and the toe at chainage 0+00. Such a failure would more than likely have taken the form of a sliding block due to the depth restrictions imposed by the sands and gravels estimated to be at elevation 1550 feet. Neither did an infinite slope type of failure provide a satisfactory description of the movements in this profile. A more suitable explanation of failure was a succession of toe circles commencing at the face of the old profile. The sketch in Figure 6.3 diagrammatically outlines the type of failure that probably occurred. The slide, numbered (1), occurred first but movement was slow. Erosion by the river had time to remove much of the material involved in this slip before slide (2) commenced. In succession then, slide (2) occurred and after erosion and further movement slide (3) occurred and so on. Such a succession of failures would have resulted in the existing profile which in effect was a series of scarps and slumped material. Old scarps are indicated in Figure 6.5 and these can be matched with scarps along the A.R.C. cutline in the stereo pair at Figure 6.1. It was also evident that due to the meander in the river, the west bank at the bridge site was being eroded rather severely. This meander appeared to be moving in a westerly direction. The east bank was relatively stable since erosion was not taking place along this shoreline.

The direction of the movement was difficult to determine. In the author's opinion the centre line of the

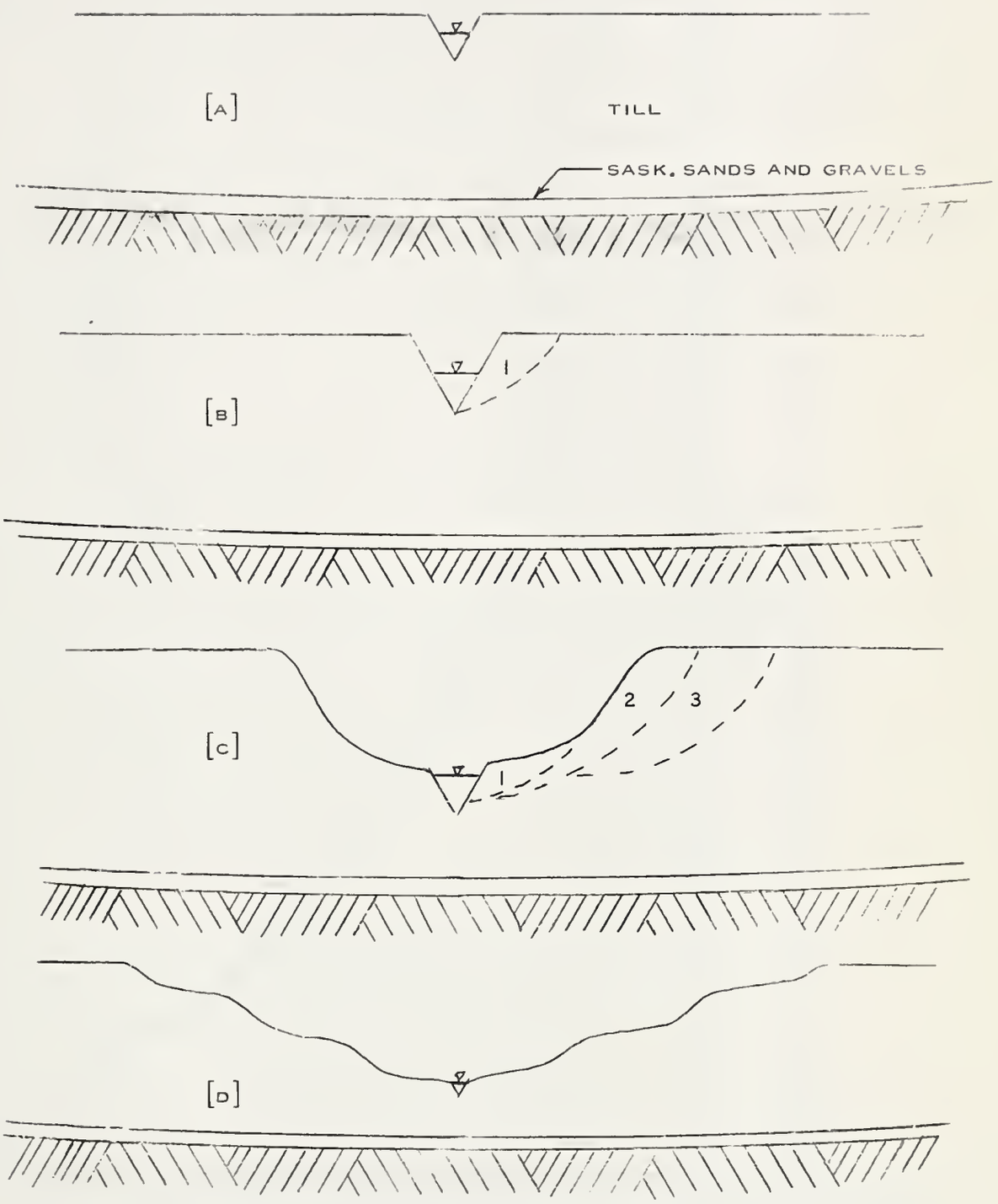


FIGURE 6.3 PROPOSED MODE OF PREVIOUS SLUMPING AT THE LITTLE SMOKY BRIDGE SITE

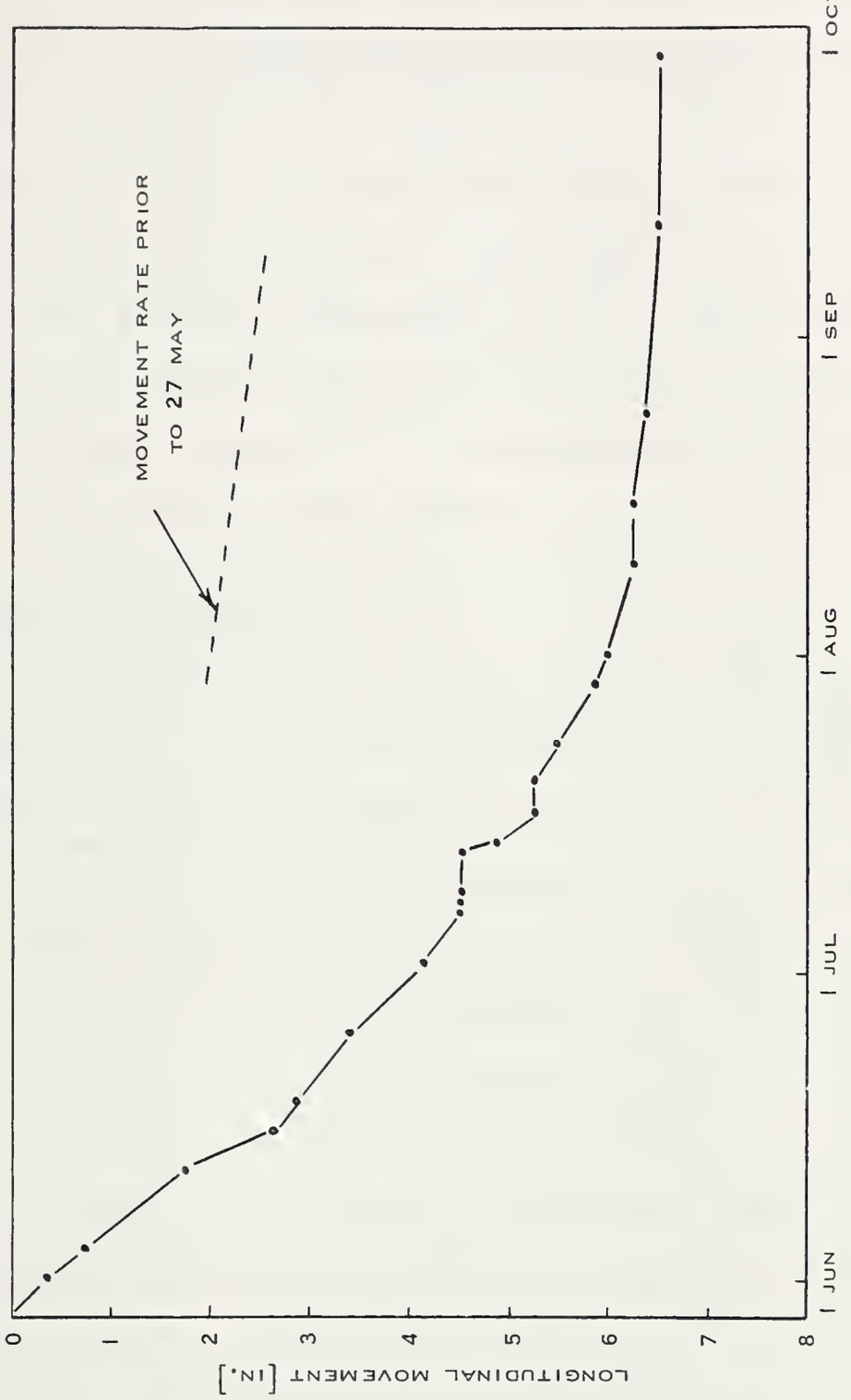
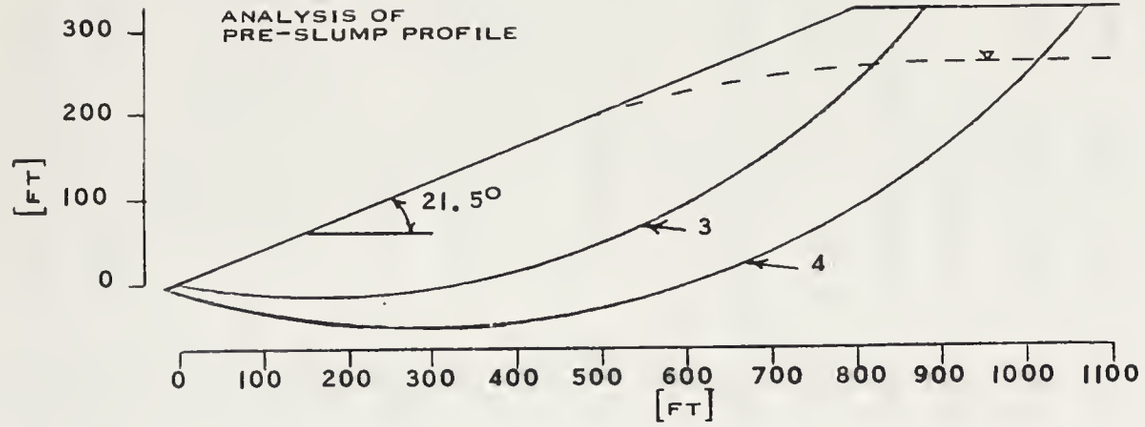


FIGURE 6.4 LONGITUDINAL MOVEMENT PIER I -1965- [DEPARTMENT OF HIGHWAYS, ALBERTA]

movement was approximately parallel to the A.R.C. cutline (Figure 6.1). The toe of the slide would then have extended from about the bridge centre line to a point approximately 500 feet north along the shoreline. This inferred that abutments 1 and 2 and pier 1 were positioned on the southern edge of the mass in motion. The combination of lateral and horizontal movement observed in pier 1 approximately coincided with the direction of the A.R.C. cutline. Slope stakes have recently been installed by the Department of Highways of Alberta to determine the direction of movement.

Stability Analyses

From the analysis mentioned concerning the slumping activity that had occurred in the past, the author felt that a stability analysis should be made on the old profile. The elevation of the top of the valley slope was considered to be at 1895 feet with the river level at 1570 feet approximately (Figure 6.5). The average slope of the old scarps observed in Figure 6.5 was somewhere between 21 and 22 degrees. After slide (1) occurred in Figure 6.3, it was probable that the face of the remaining embankment was at this angle. The analysis was then made for an embankment height of 325 feet (1895 - 1570) having a slope of 21.5 degrees with the ground water about 65 feet below the surface which was the elevation observed in borehole 6, drilled at the top of the valley slope (Appendix E). Two trials for toe



SLIP	METHOD	F	REMARKS
1	SWEDISH SLICES	1.24	OBSERVED G.W.L.
1	SWEDISH SLICES	1.04	G.W.L. AT SURFACE
2	SWEDISH SLICES	1.06	OBSERVED G.W.L.
3	SWEDISH SLICES	0.6	OBSERVED G.W.L.
4	SWEDISH SLICES	1.03	OBSERVED G.W.L.
5	INFINITE SLOPE = 12° DEPTH = 25 FT	1.00	G.W.L. 26 FT ABOVE SLIP SURFACE

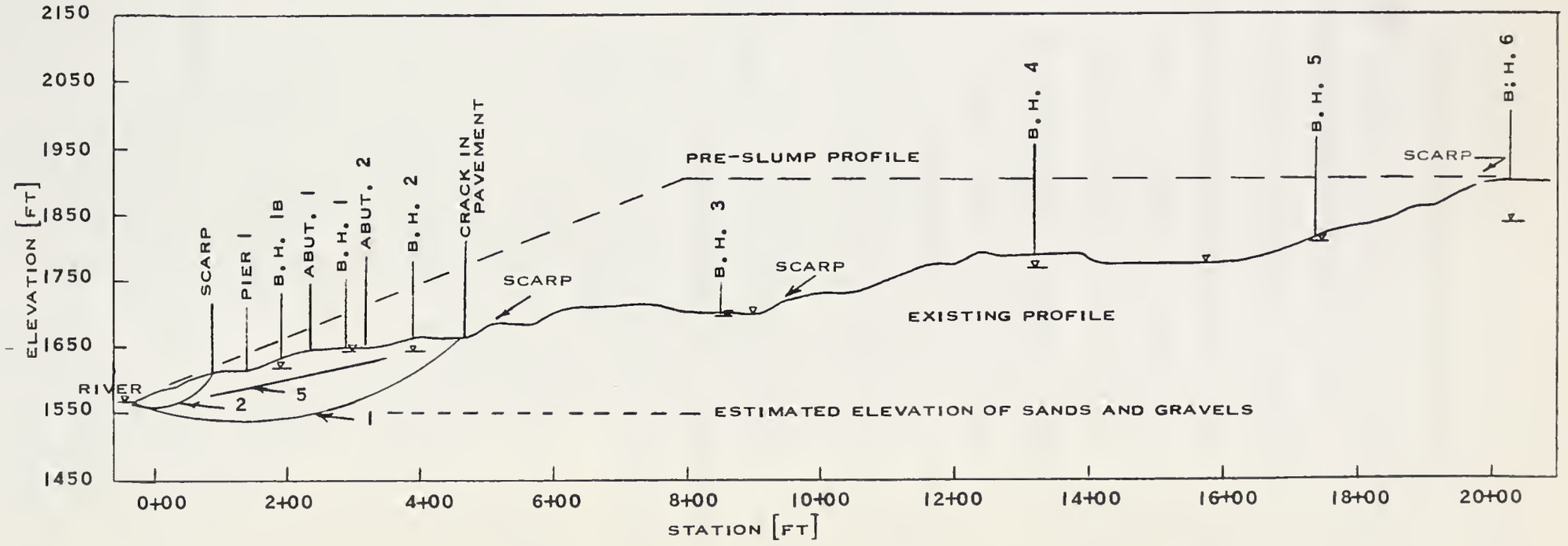


FIGURE 6.5 STABILITY ANALYSIS ON OLD AND EXISTING PROFILES AT LITTLE SMOKY RIVER BRIDGE SITE

circles using the Swedish slices method gave factors of safety of 0.6 and 1.03. The height used for the embankment analyzed was likely the maximum that ever occurred. It was also possible that the residual strength parameters were not representative of the strength existing when failure occurred many years ago but were too low. However, the low factor of safety of 0.6 indicated that it was still possible to have such a failure for an embankment having a smaller height and for conditions of higher mobilized strength.

Two circular slips were analyzed in the vicinity of pier 1 and the two abutments using the Swedish slices method. The results were:

a. Factor of Safety = 1.06: scarp 50 feet east of pier 1 as observed, toe in the river between the shoreline and the first pier in the water (Figure 6.2), pore pressures estimated from waterlevels observed in borehole 1.

b. Factor of Safety = 1.24: scarp in highway 150 feet west of abutment 2 as observed, toe in river between shoreline and the first pier in the water (Figure 6.2), pore pressures estimated from water levels in boreholes 1 and 2.

c. Factor of Safety = 1.04: same failure as detailed in sub-paragraph b. except piezometric profile coincided with the ground surface.

These failures were analyzed using the existing profile. If these were old slips it may have been better to undertake the analysis using the profile prior to failure.

Reconstruction of this profile would have resulted in an increase in the activating forces and the factor of safety would have been lower. Also, as the slide closest to the shoreline continues to move, the toe of the large slide that passes under pier 1 and the two abutments will be gradually unloaded precipitating more movement along this slip surface.

An infinite slope analysis was undertaken for a 12 degree slope with the failure plane at a depth of 25 feet. This analysis was made previously by Hardy et al (1962). A factor of safety of unity was obtained with the piezometric profile 26 feet above the failure plane. It can be seen from Table 6.1 that the required pore pressures were never measured in the piezometer installations. However, the available evidence indicated that this type of failure was not likely occurring.

Pore Pressure

The author was of the opinion that fairly high pore pressures were developed in the slope. The reasons were:

a. Very soon after heavy rains, the rate of movement of pier 1 increased considerably as illustrated in Figure 6.4. This was indicative of a rapid increase in pore pressure and/or increased toe erosion by the river.

b. Ponded water has been observed at the bases of

various scarps within the area of movement. In particular, ponds existed permanently along the A.R.C. cutline in the vicinity of boreholes 3 and 5 (Figure 6.5).

c. Although not investigated, it was believed that artesian pressures may exist in the Saskatchewan Sands and Gravels. In the opinion of the author, other sand deposits in the till formation were not likely continuous.

The slide analyzed, that had a factor of safety equal to 1.24, passed slightly below elevation 1550 feet, the estimated elevation of the Saskatchewan Sands and Gravels. It would therefore have been affected by high pore pressures in that area. The assumption made concerning the piezometric profile that gave a factor of safety of 1.04 for the slide may not have been unrealistic. One or two piezometers should be installed in the sands and gravels to confirm the presence of arterian pressures.

Summary

The main problem in connection with the stability analysis of the Little Smoky slope was the determination of the type of failure. This was difficult since no recent large scale landslide had been observed. It was then postulated that failure of the slope resulted from a succession of circular slips that proceeded back from the river. High pore pressures, toe erosion and the downward cutting action of the river initiated the slips. The movements observed in

pier 1 and the two abutments were therefore caused by continuous sliding activity along old slip surfaces in the vicinity of the bridge. Analysis of two old slips under the pier and abutments gave factors of safety of 1.06 and 1.24 using observed ground water conditions. The factor of safety of 1.24 became 1.04 if the piezometric profile was assumed to coincide with the ground surface. This assumption appeared to be valid since there was good evidence that high pore pressures existed in the slope. It could then be concluded that the residual strength parameters ($c' = 0$, $\phi'_r = 22.6$ degrees) were representative of the shearing resistance mobilized in the bank.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

Summary

In Chapter 2, a brief review was made of the current strength relationships that were expected to be found in overconsolidated clays. Subsequent to this, the previous methods used in analyzing slope stability in overconsolidated clays were described and particular emphasis was made of the apparent decrease in the shear strength of such clays over the long term. The residual strength concept and related theories were then outlined in detail and a comment was made concerning some of the pertinent physical-chemical aspects. In Chapter 3, the geology of the Little Smoky site was investigated in order to determine the soil type that was being tested. The following chapters were then devoted to a description of the laboratory test apparatus, and a discussion of the laboratory results. A stability analysis was then made of the movements at the bridge site on the Little Smoky River.

Geology

From the evidence presented in Chapter 3, which dealt with the geology of the Peace River district, it was

concluded that samples taken from the site could be classified as a clayey till. The complete profile of the bank at the bridge site was envisaged as being lacustrine clays overlying till, followed by Saskatchewan Sands and Gravels and bedrock. Due to the comparatively great depth of the Sands and Gravels and bedrock, it was concluded that the seat of the instability in the bank was in the till formation.

Soil Properties

The Little Smoky Clay Till was found to be medium to highly plastic having a well-graded grain-size distribution. The comparatively large percentage of coarse grain sizes was likely responsible for the high residual angle of internal friction obtained, 22.6 degrees. It was also determined that the predominant cation and clay mineral was calcium and montmorillonite respectively. High swelling has not previously been associated with the calcium cation and it was concluded that in this case, swell was principally due to the hydration of the clay particles and capillary effects, which make a large contribution at low initial moisture contents.

Little Smoky Clay Till was apparently subjected to a much larger load in its geologic past than exists in situ at the present time. Evidence in support of this was dilation in shear, low values of $\bar{A}_f$, the characteristic curvature of stress path plots, moisture content - log

pressure relationships, and the geologic history of the area. However, the high strength expected to be imparted by preconsolidation was absent and there was a tendency for the clay till to act normally consolidated in shear. Previous work has indicated that this may be a characteristic of other overconsolidated clays in Alberta, especially the Edmonton formation. It was suggested that the clay mineralogy and cation type may be particularly responsible for this loss of strength over the long term since cracks and fissures, which would allow softening through seepage, were not a feature of this soil type.

The loss of the effects of preconsolidation explained the comparatively high values of $\bar{A}_f$ obtained in the triaxial undrained compression tests (all were greater than zero except for two tests). It was also suggested that the high $\bar{A}_f$ values indicated underconsolidation or the presence of cementation between particles. The high strength found at test pressures less than 2.2 kg/cm^2 was likely a reflection of a developed structure that had not been broken down by the consolidation phase.

Moisture content profiles through triaxial and direct shear test undisturbed specimens indicated dilation was occurring in the failure zones or that moisture migrated toward the failure zones during shear. However, negative pore pressures at failure were not measured except in the case of two undrained triaxial compression tests and the

overall specimen volume increased for only one direct shear test. The conclusion could be drawn from the triaxial tests, that the strain rate was too high to allow for the measurement of negative pore pressures at failure at the base of the specimens. Also, there was very little deformation of the undisturbed triaxial specimens prior to failure, which supported this conclusion. Therefore, the strength parameters obtained from the triaxial tests were likely too high. But, peak triaxial and direct shear residual strengths agreed very well and there was no reason to believe that the latter did not represent fully drained strengths. This anomaly between the observations of dilation, positive pore pressures and normally consolidated characteristics could not be explained but appeared to be related to strain rate and non-uniform deformations, a field in which more research is required.

Residual Strength

The results indicated that the normally consolidated portion of the peak strength Mohr envelope (pressures greater than 2.2 kg/cm^2), as determined from the triaxial compression tests, closely agreed with the envelope that represented residual strengths obtained from the drained direct shear tests. The drop in strength from the peak to the residual value at pressures less than 2.2 kg/cm^2 was pronounced. For a soil type that retained high strength as the result of a high geological prestress, the difference between the peak

and residual strengths would have greater significance and the concept would be more effective in explaining instability. For the Little Smoky Clay Till, however, overconsolidation appeared to be a reflection of the in situ overburden pressures and therefore peak and residual strengths were represented by the same parameters, namely $c' = 0$ and $\phi'_r = 22.6$ degrees.

The comparatively high residual angle of internal friction obtained was attributed to a large percentage of coarse sizes and the lack of a high degree of preferred particle reorientation as observed in thin sections.

It was postulated that the mode of failure of the bank at the bridge site was a series of circular slips that progressed back from the river over a geologic time period. These failures were caused by the downward cutting action of the river into the till, by bank erosion, and pore pressures. Recent movements of the bridge abutments and piers were the result of sliding along old slip surfaces. The mobilized strength along the slip surfaces was then the residual strength. Analyses undertaken for two old slips gave factors of safety of 1.06 and 1.24 using observed pore pressures. The factor of safety of 1.24 changed to 1.04 if the water table was assumed to be at the ground surface. The assumption of high pore pressures did not seem unreasonable since the rate of movement of pier 1 increased after extended wet periods; ponded surface water of a permanent

nature was in evidence throughout the bank and there was a possibility of artesian pore pressures developing in the Saskatchewan Sands and Gravels.

Laboratory Procedures

The following conclusions were made concerning the test apparatus and procedures used:

a. The formula developed by Gibson and Henkel (1954) for calculating the time to failure in drained direct shear tests was considered invalid since $c_b \neq c_v$, deformations were non-uniform and shear and normal stresses were not uniformly distributed. The net effect of these factors was to make the peak strengths obtained from the direct shear tests higher than those obtained from the triaxial compression tests. The residual strengths were considered satisfactory since there was a greater time lapse before their mobilization and stresses were close to being effective. It was concluded that a trial and error procedure was necessary to obtain the correct time to failure for the direct shear box if peak strengths are required.

b. The stress reversals used in the direct shear box procedure were not ideal since the established trend between shear stress and deflection was broken with each reversal. The selection of residual strength was therefore difficult. Shear in one direction only, if possible, was considered to be the best procedure.

c. The actual area of the specimen subjected to shear in the direct shear tests was unknown principally because adhesion between the lips of the shear box and soil was likely mobilized to some extent. A large range of calculated residual shear stresses was possible. These varied from the use of the initial area to the area corrected for deflection, which was large when the residual strength was mobilized. It was then apparent that if specimens are to be sheared out to large deflections, the lips of the shear box should be made as narrow as possible to facilitate the determination of the stressed area.

d. It was found that the Mohr strength envelope, as determined from undrained triaxial compression tests on remoulded specimens, normally consolidated from the liquid limit, had a significantly larger angle of internal friction than the residual strength envelope. Therefore, this was not a satisfactory method of determining the residual strength parameters although there was evidence in the literature that better results could be obtained in the medium and high pressure ranges.

e. A comparison of the peak strength Mohr envelopes for undisturbed specimens obtained from triaxial undrained compression tests conducted in 1962 and 1965 showed that a lower strain rate reduced strength in terms of effective stress. The contradiction between the positive pore pressures measured and dilation indicated that the strain rate used

in 1965 tests should have been lower still. Since the consolidation characteristics indicated a test duration of about one working day, it appears that this method of estimating failure times for stiff clays is not reliable.

Recommendations

The following recommendations have arisen as a result of this research:

a. There was some question as to whether the undisturbed direct shear and triaxial compression test results were representative due to the likelihood that strain rates were too fast. It is therefore recommended that a strain rate to ensure a fully drained test be found for the direct shear apparatus using a trial and error procedure and Little Smoky specimens. The residual strength parameters should then be compared to those obtained in this thesis. For the triaxial undrained tests, a trial and error procedure should also be used to determine the absolute minimum values of effective stress parameters possible. This may involve a strain rate equivalent to creep with pore pressure measurement.

b. An attempt should be made to accurately determine the higher preconsolidation load of the Little Smoky Clay Till. Consolidation tests on undisturbed and remoulded specimens should be conducted using pressures up to 100 kg/cm^2 to completely define the pressure-void ratio characteristics.

High pressure triaxial tests should then be conducted in the normally consolidated and overconsolidated range to investigate the characteristics of the Mohr envelope for undisturbed specimens and to verify the strength parameters obtained. High pressure triaxial tests should also be conducted on remoulded specimens in order to investigate a curvature in this Mohr envelope and to determine a suitable pressure range in which remoulded and residual strengths agree.

c. The residual strength parameters for the overconsolidated clays of the Edmonton formation should be determined and used to investigate bank instability along the North Saskatchewan River in the Edmonton area.

d. The dilation and deformation characteristics of cohesive soils in laboratory tests require investigation. The author is not sure of the form this should take but recommends that thin sections should be taken from triaxial specimens, where possible, to observe changes in structure caused by shear. Also, a routine procedure of taking five or six final moisture contents vertically through triaxial specimens should be established to accurately define a degree of moisture migration and this may lead to a relationship between overconsolidation ratio and dilation. The results of this research may invalidate the measurement of pore pressures at the base of triaxial specimens of very stiff soils and indicate that dilation is not just restricted to apparently overconsolidated specimens.

e. Research should be continued into the shearing resistance of homionic montmorillonite cohesive soils, particularly overconsolidated specimens. It is suggested that a comparison of a potassium and sodium clay be made with respect to the degree of retention of cohesion and high strength after being rebounded from a high prestress. The prestress should be sufficiently high to provide test moisture contents no greater than 30 per cent after a rebound to approximately 100 psi, which would approximately duplicate field conditions found in Alberta. Normal laboratory test procedures pertaining to the triaxial compression test should be employed. It is possible that the specimens homionic to sodium may act normally consolidated in shear even though subjected to a high degree of preconsolidation.

f. With respect to the movements at the Little Smoky bridge site, it is suggested that slope drainage be improved. Some form of shore protection should also be constructed on the west bank to prevent river erosion. The exact area that the protection should cover should be known after the movements have been defined by slope stakes, recently installed.

LIST OF REFERENCES

LIST OF REFERENCES

Ali, P., 1965, "The Mechanics of Swelling in Clay Soils", Unpublished M.Sc. Thesis, University of Alberta, Edmonton.

Allan, J.A., Rutherford, R.L., 1934, "Geology of Central Alberta", Scientific and Industrial Research Council of Alberta, Report No. 30.

Bayrock, L.A., Geology 466 Lecture delivered at the University of Alberta, Edmonton.

Bayrock, L.A., Hughes, G.M., 1962, "Surficial Geology of the Edmonton District of Alberta", Research Council of Alberta, Report 62-6.

Bishop, A.W., Alpan, I., Blight, G.E., Donald, I., 1960, "Factors Controlling the Strength of Partly Saturated Cohesive Soils", ASCE Research Conference on the Shear Strength of Cohesive Soils, pp 503-532.

Bishop, A.W., Bjerrum, L., 1960, "The Relevance of the Triaxial Test to the Solution of Stability Problems", Norwegian Geotechnical Institute Publication No. 34.

Bishop, A.W., Henkel, D.J., 1962, The Measurement of Soil Properties in the Triaxial Test, Edward Arnold Ltd., London

Bishop, A.W., Webb, D.L., Lewin, P.I., 1965, "Undisturbed Samples from the Ashford Common Shaft: Strength-Effective Stress Relationships", Geotechnique Vol. 15, No. 1, pp 1-31.

Bjerrum, L., DiBiago, E., Kjaernsli, B., 1957, "Triaxial Equipment Developed at the Norwegian Geotechnical Institute", Norwegian Geotechnical Institute Publication No. 21, Oslo.

Bjerrum, L., Kjaernsli, B., 1957, "Analysis of the Stability of Some Norwegian Natural Clay Slopes", Geotechnique, Vol. 7, No. 1, pp 1-16.

Brooker, E.W., 1965, Personal Communication, University of Alberta, Edmonton.

Casagrande, A., Wilson, S.D., 1951, "Effect of Rate of Loading on the Strength of Clays and Shales at Constant Water Content", Harvard Soil Mechanics Series No. 39.

Casagrande, A., Wilson, S.D., 1953, "Prestress Induced in Consolidated-Quick Triaxial Tests", Harvard Soil Mechanics Series No. 42.

Dahlman, A.E., 1964, "The Influence of Strain on the Shear Strength Parameters of a Highly Plastic Remoulded Homionic Clay Soil", Unpublished M.Sc. Thesis, University of Alberta, Edmonton.

Denissov, N.Ya., 1965, "Pore Pressure and Strength of Underconsolidated Clay Soils", Proceedings of the Sixth International Conference on Soil Mechanics and Foundation Engineering, Montreal, pp 208-212.

Elson, J.A., 1960, "The Geology of Tills", Proceedings of the Fourteenth Canadian Soil Mechanics Conference, Technical Memorandum No. 69, pp 5-36.

Flint, R.F., 1963, Glacial and Pleistocene Geology, John Wiley and Sons Inc., New York.

Gibson, R.E., Henkel, D.J., 1954, "Influence of Duration of Tests at Constant Rate of Strain on Measured Drained Strength", Geotechnique, Vol. 4, No. 1, pp 6-15.

Hardy, R.M., Brooker, E.W., Curtis, W.E., 1962, "Landslides in Overconsolidated Clays", The Engineering Journal, June.

Henderson, E.P., 1958, "Surficial Geology Sturgeon Lake Map-Area Alberta", Geological Survey of Canada, Memoir 303.

Henkel, D.J., 1957, "Investigations of Two Long Term Failures in London Clay Slopes at Wood Green and Northolt", Proceedings Fourth International Conference Soil Mechanics and Foundation Engineering, London, Vol. 2, pp 315-320.

Henkel, D.J., Skempton, A.W., 1955, "A Landslide at Jackfield, Shropshire in an Over-Consolidated Clay", Geotechnique, Vol. 5, pp 131-137.

Hirschfeld, R.C., 1959, "The Relation Between Shear Strength and Effective Stress", Proceedings First Pan American Conference Soil Mechanics Foundation Engineering, Mexico City, pp 517-536.

Holubec, I., Scott, J.D., 1965, "A Triaxial Piston Load Transducer", Unpublished Paper, Department of Civil Engineering, University of Waterloo, Waterloo, Ontario.

Hvorslev, M.J., 1960, "Physical Components of the Shear Strength of Saturated Clays", ASCE Shear Strength Conference on Shear Strength of Cohesive Soils, pp 169-273.

Lambe, T.W., 1960, "A Mechanistic Picture of Shear Strength in Clay", ASCE Research Conference on Shear Strength of Cohesive Soils, pp 555-580.

Lambe, T.W., 1962, Soil Testing for Engineers, John Wiley and Sons Inc., New York.

Lerbekmo, J.F., 1965, Personal Communication, University of Alberta, Edmonton.

Lindberg, D.A., 1965, "Comparative Aspects of Five Piezometer Designs", Unpublished M.Sc. Thesis, University of Alberta, Edmonton.

Lowe, L.E., 1965, Personal Communication, University of Alberta, Edmonton.

Lowe, J., Johnson, T.C., 1960, "The Use of Back Pressure to Increase Degree of Saturation of Triaxial Test Specimens", ASCE Research Conference on Shear Strength of Cohesive Soils, pp 819-836.

Mitchell, J.K., 1956, "The Fabric of Natural Clays and its Relation to Engineering Properties", Highway Research Board Proceedings, pp 693-713.

Olsen, R.E., 1960, "The Migration of Pore Fluid During Shear in Consolidated-Undrained (R) Triaxial Tests", ASCE Research Conference on Shear Strength of Cohesive Soils, pp 1043-1049.

Painter, W.T., 1965, "An Investigation of LeSueur Landslide", Unpublished M.Sc. Thesis, University of Alberta, Edmonton.

Peterson, R., Jaspar, J.L., Rivard, P.J., Iverson, N.L., 1960, "Limitations of Laboratory Shear Strength in Evaluating Stability in Highly Plastic Clays", ASCE Research Conference on Shear Strength of Cohesive Soils, pp 765-791.

Ringheim, A.S., 1964, "Experiences with the Bear Paw Shale at the South Saskatchewan River Dam", Eighth International Conference on Large Dams, Vol. 1, Edinburgh, Great Britain, pp 529-550.

Rosenqvist, I.Th., 1955, "Investigations in the Clay-Electrolyte-Water System", Norwegian Geotechnical Institute Publication No. 9, Oslo.

Rutherford, R.L., 1930, "Geology and Water Resources in Parts of the Peace River and Grande Prairie Districts, Alberta", Scientific and Industrial Research Council of Alberta, Report No. 21.

Skempton, A.W., 1948, "The Rate of Softening in Stiff Fissured Clays with Special Reference to London Clay", Proceedings of the Second International Conference on Soil Mechanics and Foundation Engineering, Vol. 2, pp 50-53.

Skempton, A.W., 1954, "The Pore Pressure Coefficients A and B", Geotechnique Vol. 4, pp 143-147.

Skempton, A.W., 1964, "The Long Term Stability of Clay Slopes", Geotechnique Vol. 14, pp 77-102.

Skempton, A.W., Delory, F.A., 1957, "Stability of Natural Slopes in London Clay", Proceedings of the Fourth International Conference on Soil Mechanics and Foundation Engineering, London, pp 378-381.

Skempton, A.W., Brown, J.D., 1961, "A Landslide in Boulder Clay at Selset, Yorkshire", Geotechnique 11:4, pp 280-293.

Shockley, W.G., Ahlvin, R.G., 1960, "Non-uniform Conditions in Triaxial Test Specimens", ASCE Research Conference on Shear Strength of Cohesive Soils, pp 341-357.

Taylor, D.W., 1948, Fundamentals of Soil Mechanics, John Wiley and Sons Inc., New York.

Terzaghi, K., 1936, "Stability of Slopes in Natural Clay", Proceedings of the First International Conference on Soil Mechanics and Foundation Engineering, Cambridge, pp 161-165.

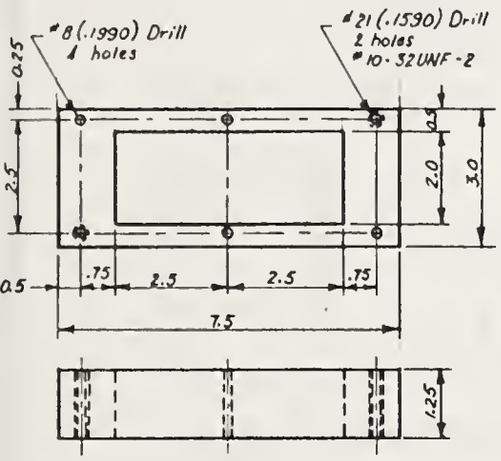
Westgate, J.A., 1965, Personal Communication, University of Alberta, Edmonton.

Wroth, C.P., 1958, "Soil Behaviour During Shear", Engineering, Vol. 186, Jul-Dec, pp 409-413.

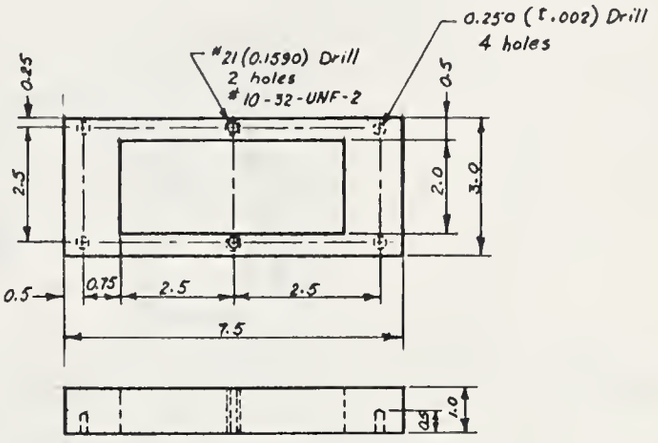
Appendix A

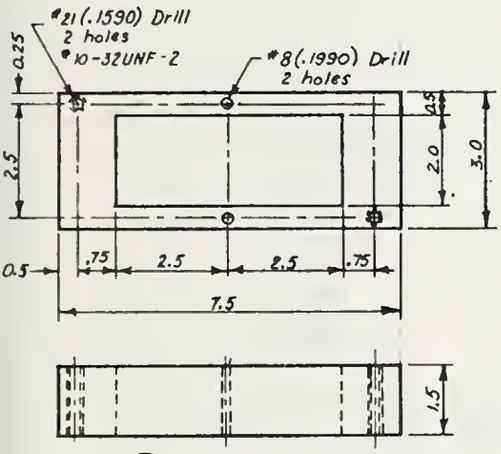
DRAWINGS OF SHEAR BOX AND TRIMMER

② UPPER BOX

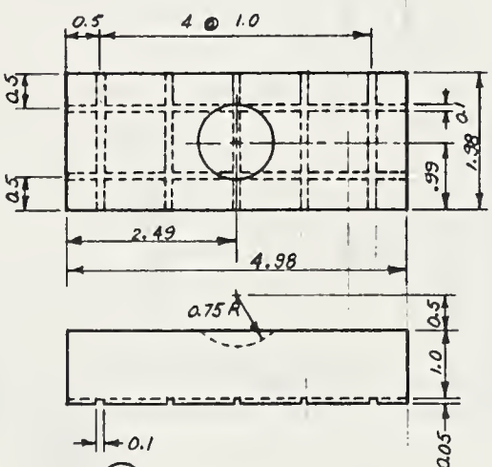


④ LOWER BOX





③ MIDDLE BOX

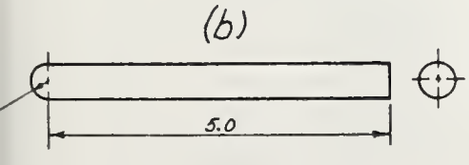


⑤ LOADING CAP

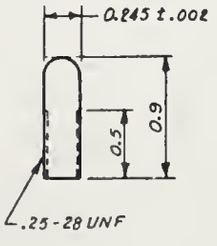
- GENERAL NOTES -

1. All dimensions in inches.
2. All materials stainless steel.
3. Remove all burrs.
4. Finish 125√ all over.

(b)



⑦ PISTON

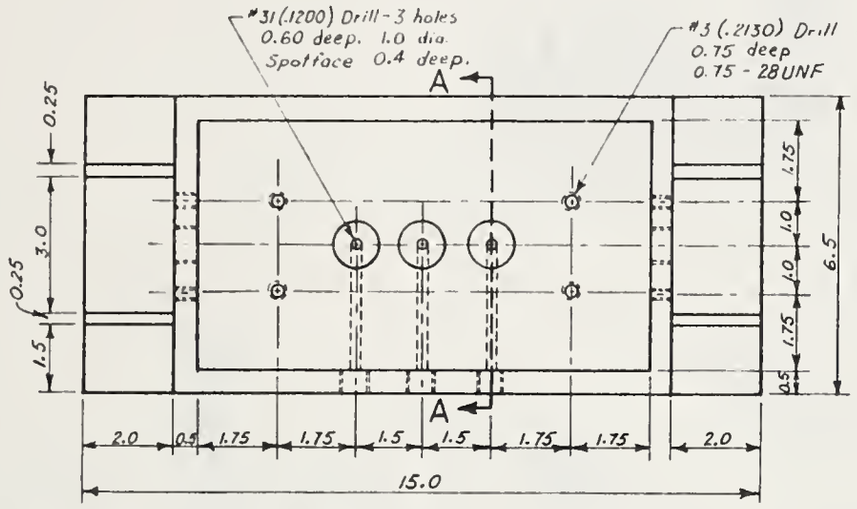


⑧ SHEAR PIN
4 REQD

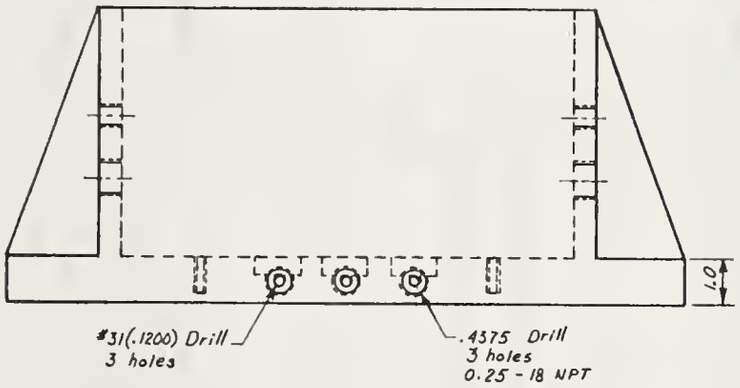
UNIVERSITY OF ALBERTA
FACULTY OF ENGINEERING

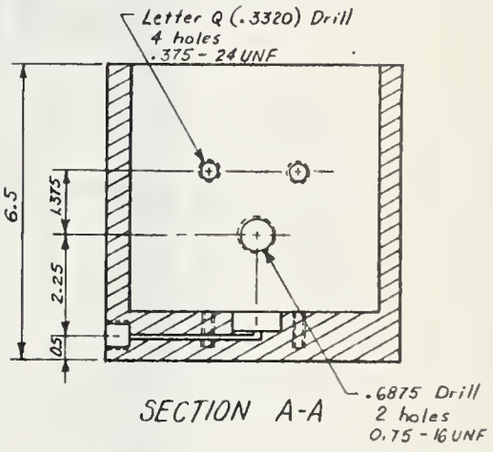
SHEAR BOX DETAIL

DR. W.W.P.	DEPT. Technical Illustrating		
CK. [illegible]	SCALE as shown	No. 1 of 2	
APP.	DATE 27/1/65		

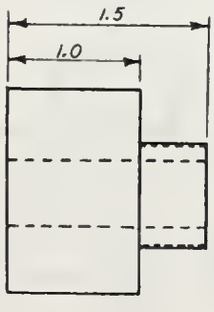


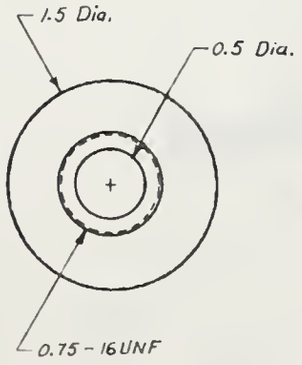
① CONTAINER



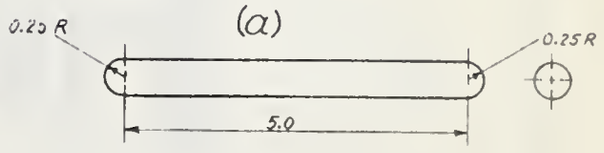


SECTION A-A

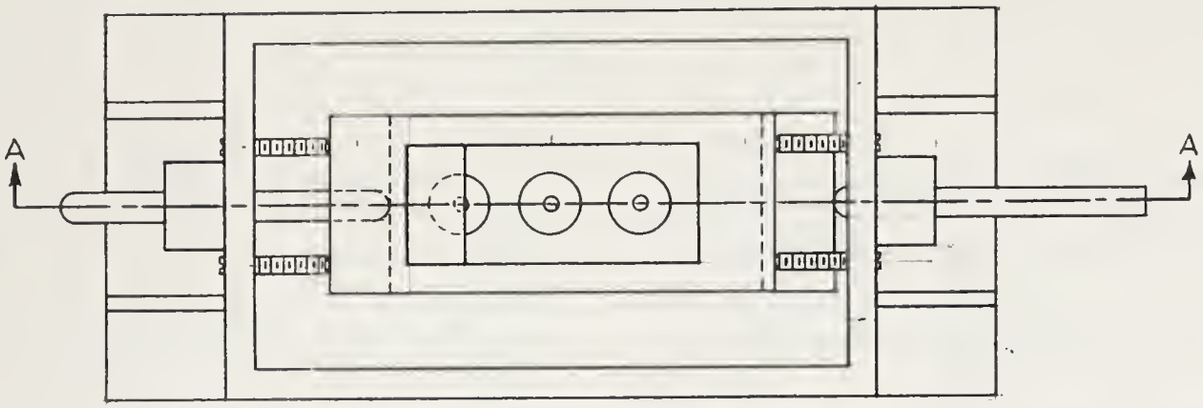


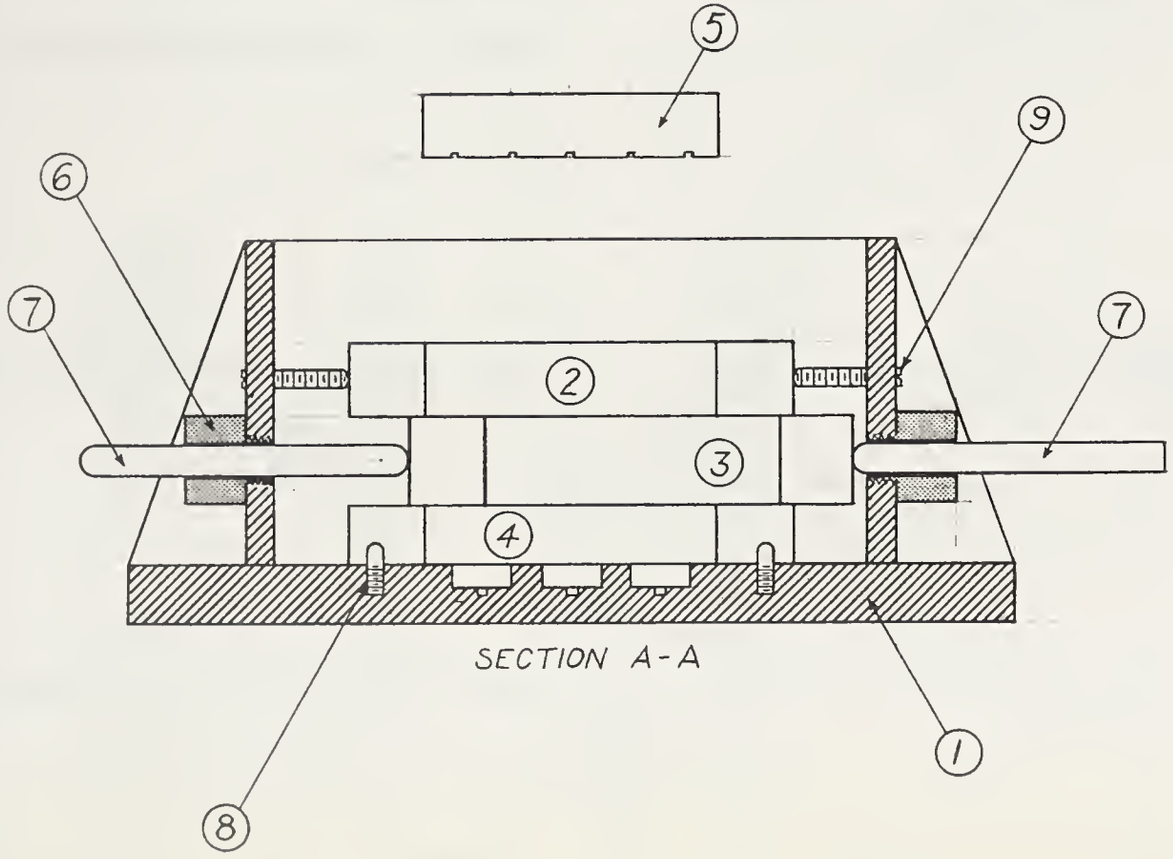


⑥ BUSHING
2 REQD



⑦ PISTON





SECTION A-A

1. Container
2. Upper Box
3. Middle Box
4. Lower Box
5. Loading Cap
6. Bushing
7. Piston
8. Shear Pin
9. Set Screw (slotted, headless, flat point, 3/8 - 24 UNF - 2 Reqd)

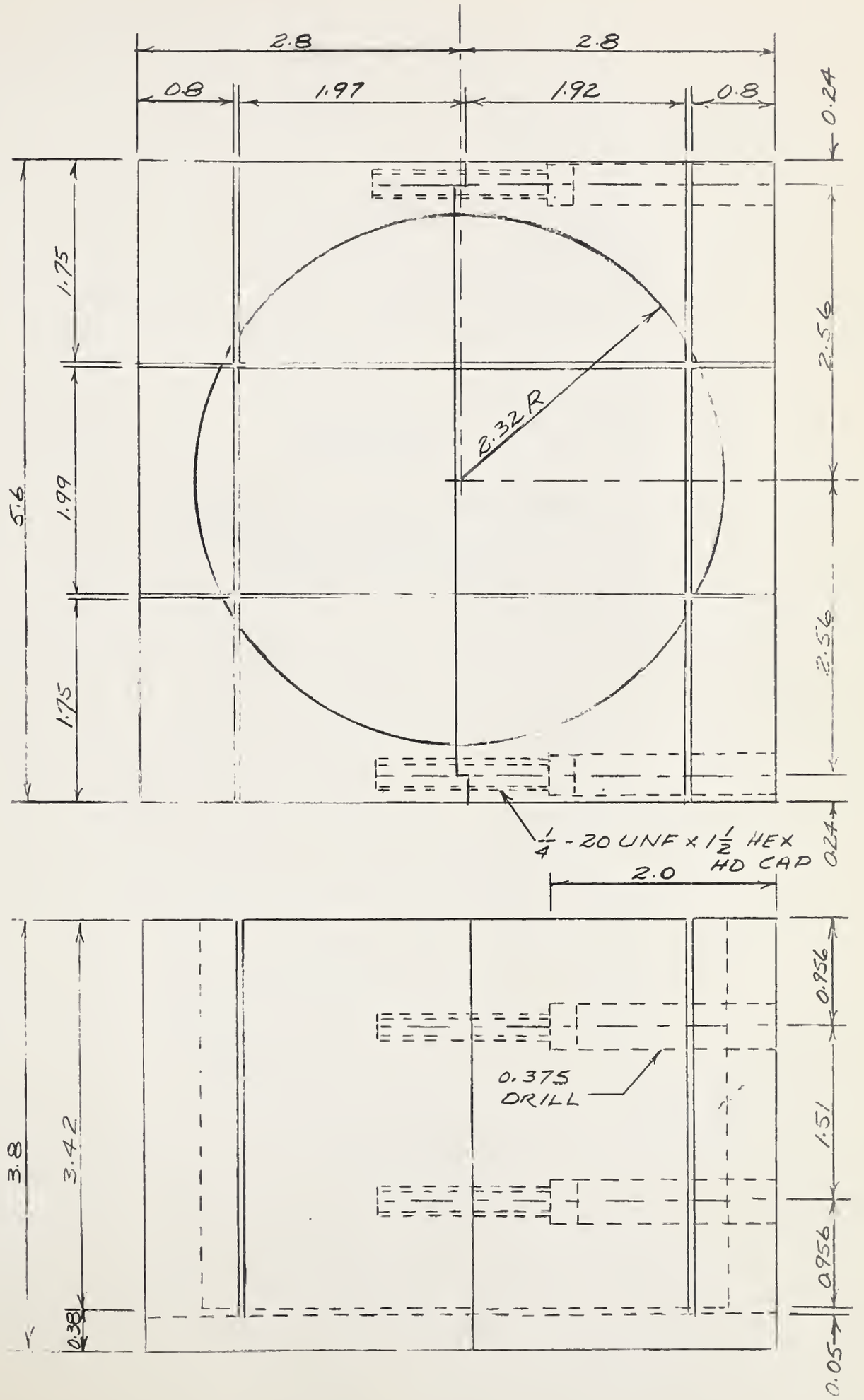
UNIVERSITY OF ALBERTA
FACULTY OF ENGINEERING

SHEAR BOX ASSEMBLY

DR. W.W.P.	DEPT. Technical Illustrating	
CK. DAP	SCALE half size	No. 2 of 2
APP.	DATE 2/3/65	

A3

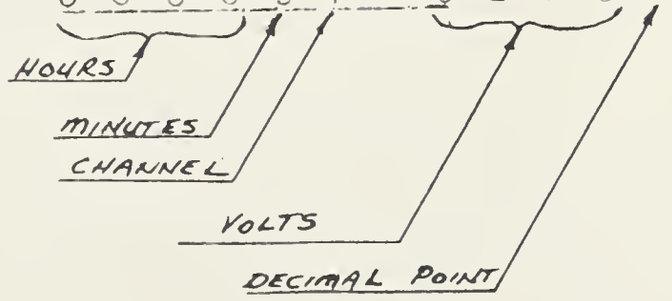
TRIMMER FOR DIRECT SHEAR BOX



Appendix B

SAMPLE OF PRINTED TAPE FROM DIGITAL RECORDER

1	4	0	0	0	2	+	1	7	0	0	3
1	4	0	0	0	1	−	0	2	5	9	5
1	3	0	0	0	6	−	0	7	3	4	5
1	3	0	0	0	2	+	1	7	1	9	3
1	3	0	0	0	1	−	0	2	6	5	5
1	2	0	0	0	6	−	0	7	6	4	5
1	2	0	0	0	2	+	1	7	4	0	3
1	2	0	0	0	1	−	0	2	6	4	5
1	1	0	0	0	6	−	0	7	6	4	5
1	1	0	0	0	2	+	1	7	6	1	3
1	1	0	0	0	1	−	0	2	6	4	5
1	0	0	0	0	6	−	0	7	6	4	5
1	0	0	0	0	2	+	1	7	8	1	3
1	0	0	0	0	1	−	0	2	6	6	5
0	9	0	0	0	6	−	0	7	6	3	5
0	9	0	0	0	2	+	1	8	0	3	3
0	9	0	0	0	1	−	0	2	7	0	5
0	8	0	0	0	6	−	0	7	6	4	5
0	8	0	0	0	2	+	1	8	2	5	3
0	8	0	0	0	1	−	0	2	7	4	5
0	7	0	0	0	6	−	0	7	6	3	5
0	7	0	0	0	2	+	1	8	4	5	3
0	7	0	0	0	1	−	0	2	7	5	5
0	6	0	0	0	6	−	0	7	6	3	5
0	6	0	0	0	2	+	1	8	6	4	3
0	6	0	0	0	1	−	0	2	7	4	5
0	5	0	0	0	6	−	0	7	6	2	5
0	5	0	0	0	2	+	1	8	8	2	3
0	5	0	0	0	1	−	0	2	7	6	5
0	4	0	0	0	6	−	0	7	6	3	5
0	4	0	0	0	2	+	1	9	0	1	3
0	4	0	0	0	1	−	0	2	7	6	5
0	3	0	0	0	6	−	0	7	6	3	5
0	3	0	0	0	2	+	1	9	1	9	3
0	3	0	0	0	1	−	0	2	7	6	5



CHANNEL 1 READING AT 0300 = − 0.00276 VOLTS

Appendix C

PROCEDURE FOR DIRECT SHEAR TESTS

PROCEDURE FOR PREPARATION OF THIN SECTIONS

APPENDIX C

MOTORIZED SHEAR BOX PROCEDURE

Preparation for Test

a. Saturate porous stones at bottom of shear box tank.

b. Place saturated filter paper over porous stones.

c. Place saturated porous stone over filter paper. Nickel porous plate is available for the 4 in. x 2 in. shear box size.

d. Place saturated filter paper over porous stone and dentated plate over filter paper.

e. Place lower shear box section.

f. Saturate the lower porous stone with water from the burettes connected to the shear box tank.

g. Deaired, distilled water should be used throughout.

Preparation of Specimen

a. A stainless steel trimmer is available to cut 4 in. x 2 in. samples from 4.75 inch Shelby tube samples. Specimens can be made any height up to 4 inches but consideration of settlement during consolidation should be made in order that the top of the sample does not settle

below the top shear plane. The ideal is to obtain a sample height that will ensure equal drainage paths from the top and bottom shear plane. This height is approximately 3 inches when the thickness of porous plates and teeth are considered. A wire saw can be used to cut the sample along the trimmer mortices.

b. Determine initial moisture contents from the trimmings as desired.

c. Measure the width, height and length of the specimen and weigh.

d. Place the specimen in the lower section of the shear box and press lightly onto the dentated plate.

e. Place the remaining two shear box sections over the sample.

f. Insert bolts into the shear box. In this order, place dentated plate, filter paper, porous plate, load cap and ball bearing on top of the sample.

g. Tighten the set screws of the top shear box section.

Consolidation

a. Determine the rate of loading to be used in the consolidation phase. It must be remembered that the tare pressure of the hangar and lever arm is 0.914 Tsf for a 4 in. x 2 in. sample and this does

not include the weight of the porous stone and ball bearing.

b. To start consolidation, place the load hangar and weights and counterbalance the lever arm. Fill the shear box with distilled water. Adjust a dial gauge or DTDT over the hangar to record vertical deflections. Counterbalance the lever arm with the screw jack as volume changes occur.

Shear Test

a. Determine the coefficient of consolidation C_b from the consolidation phase in the shear box. The square root time method (Taylor, 1948) appears to give the best time-settlement relationship. Using this value of C_b, calculate the time to failure using the formula derived by Gibson and Henkel (1954), $t_f = \frac{H^2}{2\ C_b\ (1 - u_c)}$ where H is one half the height of the sample and u_c is the degree of consolidation usually taken as 95 per cent.

b. Insert the load cell and adjust the dial gauge or DTDT to measure the horizontal deflection. Select the gear settings to give the appropriate time to failure. An estimate of the failure deflection must be made at this time. A deflection of 0.1 inch at failure has been found for a Lake Edmonton Clay trial test and the Little Smoky Clay Till. The

deflection at failure for softer soils will have to be found by trial. The gear settings are as follows:

1. 45/45 gear ratio (1:1)

   Setting A .0086 in/min.

   Setting B .00172 in/min.

   Setting C .000345 in/min.

   Setting D .000069 in/min.

   Setting E .0000l4 in/min.

2. 60/30 gear ratio (1:2)

   All settings 2 times (45/45) rates.

3. 30/60 gear ratio (2:1)

   All settings one-half of (45/45) rates.

4. 54/36 gear ratio (1:1.5)

   All rates 1.5 times (45/45) rates.

5. 36/54 gear ratio (1.5:1)

   All rates 1/1.5 of (45/45) rates.

Separate slightly the shear box sections with the lift pins and remove. Seat the load cell and drive piston by bringing on the load slightly with a high rate of deformation. Set the gear ratio at the desired setting and start the shear test. Periodically counterbalance the lever arm as volume changes occur.

c. To reverse the direction of shear, turn off the

motor, reverse the motor and switch to a high deformation rate to release the load. Remove the instrumentation and disconnect the bolts holding the tank to the platform. Rotate the tank 180 degrees. This can be done without removing the normal load if shorter pistons are inserted into the ends of the tank. Bolt the tank to the platform, insert pistons and adjust instrumentation. Start the shear test as previously described. This operation can be carried out in approximately 10 minutes.

d. After completion of the shear test, drain the tank by means of a siphon. Then remove the shear force. This will ensure that a minimum of water will be adsorbed by the specimen after the removal of the shear stress. Remove the normal load and sample. Weigh the sample and take moisture contents, preferably along the failure lines.

PROCEDURE FOR THIN SECTIONS

Theory

1. When thin sections are taken from undisturbed natural clays, it is essential that at least two sections be made at right angles to each other. Since clay particles have been described as having thin rectangular shapes, it is possible that one thin section may show no orientation when rotated in polarized light even though preferred orientation is present. This is true if the line of sight is perpendicular to the broad face of the clay particles. For particles shaped like rods, it is also obvious that if the line of sight is parallel to the long axes of the rods, that no distinction in illumination will be observed when rotated in plane polarized light. Taking sections at right angles ensures that at least one section should show preferred orientation if it is present (Mitchell, 1956).

2. In the direct shear tests conducted, the author assumed that the long axes of clay particles would be horizontal and oriented in the direction of shear. It was also assumed that plate-like particles would be oriented with their broad surfaces facing upward. Only one thin section was therefore taken, which was vertical and parallel

to the direction of shear.

3. The procedure used was as follows:

   a. A specimen having the approximate dimensions of 1/4" x 1" x 2" was taken and submerged in melted Carbowax heated to 150°F.

   b. The specimen was left in the Carbowax for three days at 150°F. During this period the Carbowax and pore water formed uniform mixture by diffusion. This procedure eliminates to a great extent the structural disturbance caused by capillary tensions during drying (Mitchell, 1956).

   c. The specimen was then allowed to cool at room temperature.

   d. One side of the specimen was ground to a smooth, flat surface using kerosene as a coolant and carborundum powder.

   e. The specimen was then mounted on a heated slide (ground side in contact with the slide) Lakeside 70 glue (Refractive index = 1.54) was used which required heating. When applying the glue to the slide, an effort was made to remove foreign objects and air bubbles.

   f. After the glue had cooled, the slide and specimen were affixed to a grinding clamp.

   g. A mechanical grinder was used for grinding

with kerosene as a coolant and carborundum powder. The thickness obtained by this operation was controlled by the grinding clamp.

h. Final grinding was conducted by hand. The specimen was rubbed over a glass plate covered with carborundum powder with kerosene as a coolant. As the specimen became thinner, the birefringeant colour of the quartz grains changed in the order of cloudy, dark green, blue, red and yellow under crossed nicol prisms. A uniform specimen thickness was obtained by ensuring quartz grains throughout the specimen had the same colour as grinding progressed. When all quartz grains had a yellow colour, the specimen was thin enough for rotation in polarized light.

j. The specimen was then covered with Permount (Fisher Scientific Co. Ltd.) for protection. The Permount was allowed to cool.

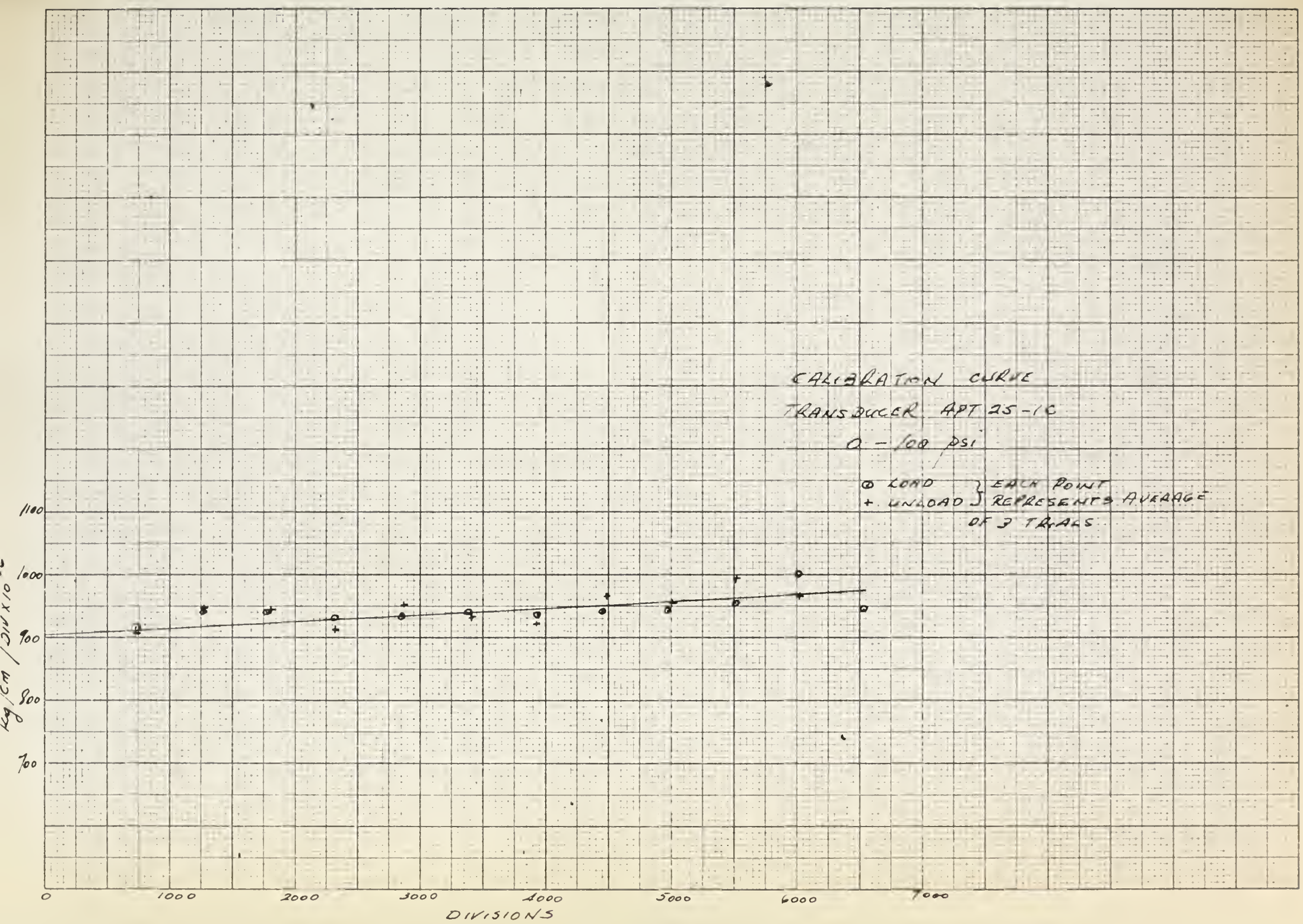
k. The specimen was then rotated under polarized light, various light intensities being used to achieve the greatest distinction in illumination.

4. Carbowax is produced by Union Carbide Ltd., and is available at different melting points. Carbowax 20,000 was found to be successful since it had a high

melting point and the wax did not melt when the specimen was mounted on the heated slide. Mitchell (1956) used Carbowax 6000 which has a lower melting point but he used glue that required no heat.

Appendix D

CALIBRATION CURVE FOR TRANSDUCER


CALIBRATION CURVE
TRANSDUCER APT 25-1C
0 - 100 PSI
⊙ LOAD
+ UNLOAD
EACH POINT REPRESENTS AVERAGE OF 3 TRACES
1100
1000
900
800
700
Kg/CM² (SW x 10⁻³)
0
1000
2000
3000
4000
5000
6000
7000
DIVISIONS

Appendix E

BOREHOLE LOGS

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
SOIL PROFILE AND SUMMARY OF LABORATORY TESTING

PROJECT	LITTLE SMOKY INVESTIGATION
SITE	
LOCATION	
HOLE	ARC 1
TECHNICIAN	RESEARCH COUNCIL OF ALTA.
DATE	26 OCT 1960

PLOT OF ATTERBERG LIMITS & NATURAL MOISTURE CONTENTS

10 20 30 40 50 60 70 80 90%

DEPTH FT	SAMPLE	GROUND SURFACE	NUMBER OF BLOWS	MOIST. CONT. %	UNC. COMP. STR. T/SQ FT.
		CLAY ORGANIC			
2					
4					
	4-1				
6					
8		SAND POCKET			
10	4-2				
12		MEDIUM TO HIGH PLASTIC, DARK GREY			
14					
	4-3				
16					
18					
20	4-4				
22		5 6			
24		7 8			
26					
28					
	4-1A	TRACES OF SILT AND SAND			
30		G.W.L			
	4-2A				
32					
34					
36					
38					

NOTE;
- ⊡ PLASTIC LIMIT
- △ LIQUID LIMIT
- ⊙ NATURAL MOISTURE CONTENT

G. W. L. INDICATES GROUND WATER LEVEL AT TIME TEST HOLE WAS DRILLED

NUMBER OF BLOWS REFERS TO THE NO OF BLOWS OF A 30 LB. RAMMER DROPPED 30 INS REQ. TO DRIVE A 3" DIAM. TUBE A DIST. AS INDICATED INTO SOIL

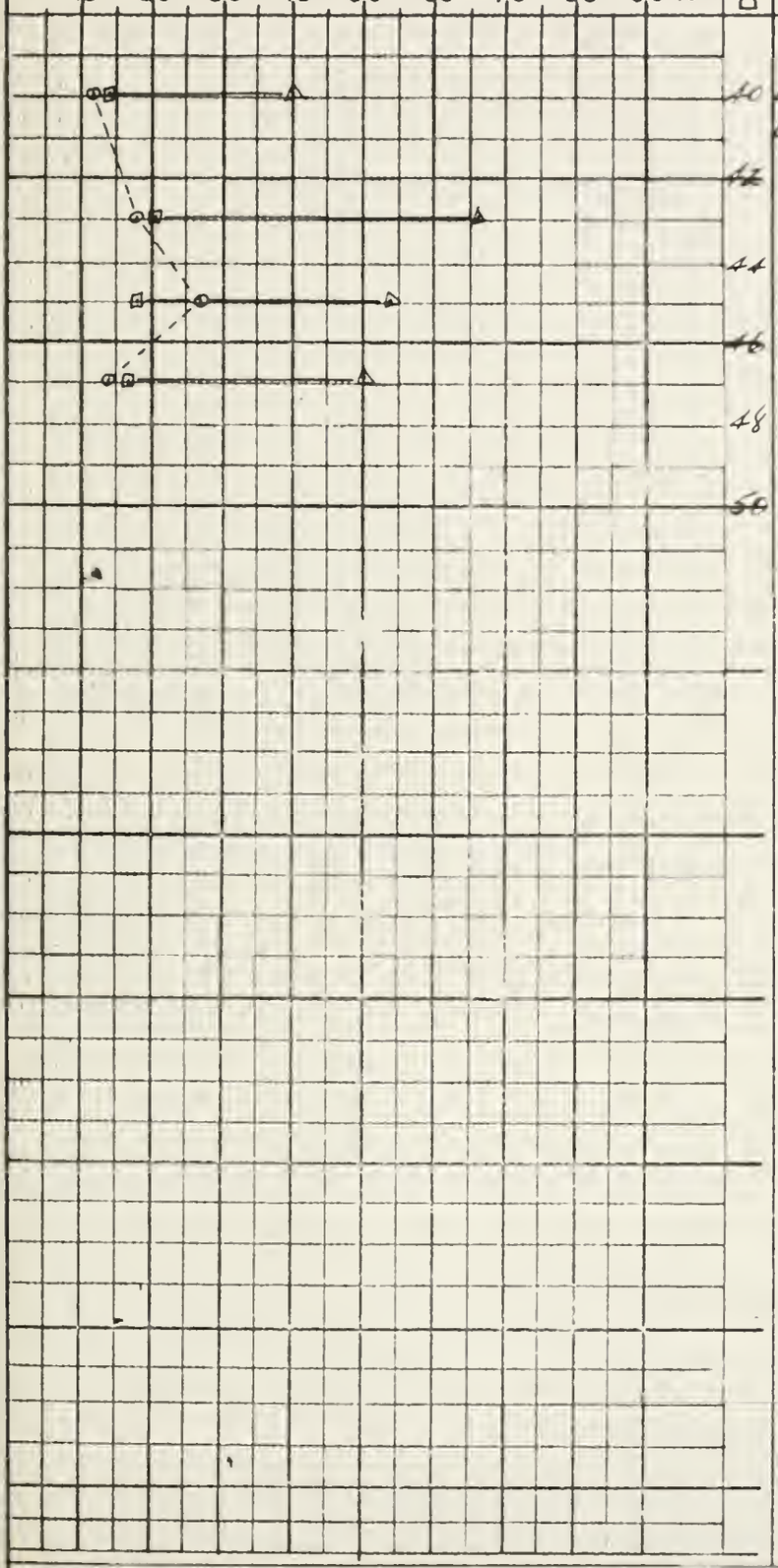
E2

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
SOIL PROFILE AND SUMMARY OF LABORATORY TESTING

PROJECT	
SITE	
LOCATION	
HOLE	ARC 1
TECHNICIAN	
DATE	26 OCT 1960

PLOT OF ATTERBERG LIMITS & NATURAL MOISTURE CONTENTS

10 20 30 40 50 60 70 80 90%



DEPTH FT	SAMPLE	GROUND SURFACE	NUMBER OF BLOWS	MOIST. CONT. %	UNC. COMP. STR. T/SQ.FT.
40	4-40				
	4-40				
42					
44					
46		VERY DENSE AND DRY			
48					
50		HOLE ADVANCED TO 125 FT THROUGH DRY SHALE AND SANDSTONE.			
		NOTE: SAMPLES NOT SUITABLE FOR STRENGTH TESTS.			

NOTE;
- ⊡ PLASTIC LIMIT
- △ LIQUID LIMIT
- ⊙ NATURAL MOISTURE CONTENT

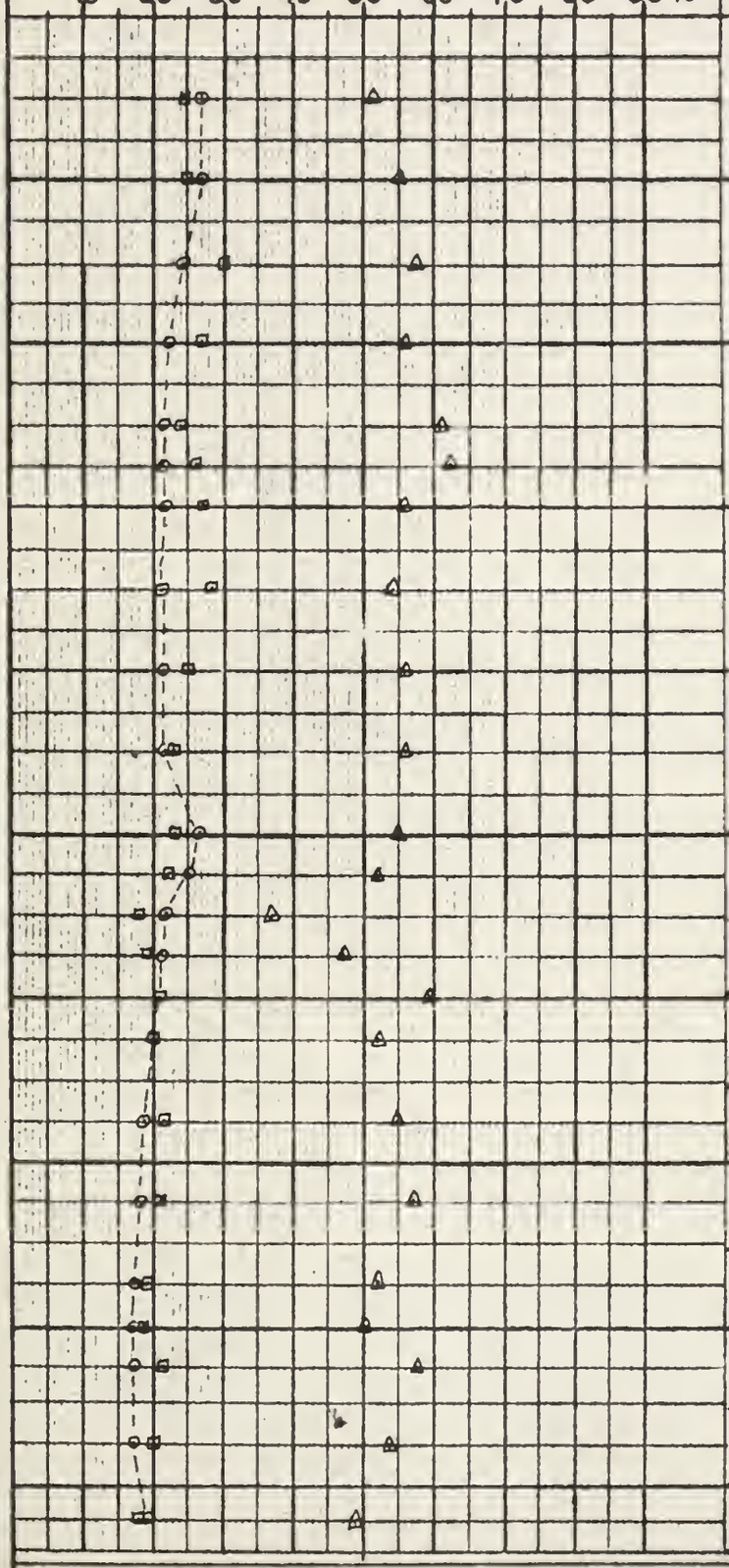
G. W. L. INDICATES GROUND WATER LEVEL AT TIME TEST HOLE WAS DRILLED

NUMBER OF BLOWS REFERS TO THE NO. OF BLOWS OF A 30 LB. RAMMER DROPPED 30 INS REQ. TO DRIVE A 3" DIAM. TUBE A DIST. AS INDICATED INTO SOIL

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
SOIL PROFILE AND SUMMARY OF LABORATORY TESTING

PROJECT LITTLE SMOKY INVESTIGATION
SITE
LOCATION
HOLE ARC 1B
TECHNICIAN RESEARCH COUNCIL OF ALTA
DATE 1 NOV 1960

PLOT OF ATTERBERG LIMITS & NATURAL MOISTURE CONTENTS
10 20 30 40 50 60 70 80 90 %



DEPTH FT	SAMPLE	GROUND SURFACE	NUMBER OF BLOWS	MOIST. CONT. %	UNC. COMP. STR. T/SQ.FT.
		CLAY			
2		SILTY			
4		TRACE OF SAND			
	U-1				
6		BROWN GREY			
8					
10		VERY DENSE			
	U-2				
12		DARK GREY			
14					
	U-3				
16					
18					
		MOIST			
20					
	U-4				
	5	SAND SEAM G.W.L			
22	6				
	7	DRY, DENSE			
24	8	DARK GREY			
26					
28					
30	U-9				
	10				
32		VERY DENSE			
34					
36					
38					

NOTE;
- ⊡ PLASTIC LIMIT
- △ LIQUID LIMIT
- ⊙ NATURAL MOISTURE CONTENT

G. W. L INDICATES GROUND WATER LEVEL AT TIME TEST HOLE WAS DRILLED

NUMBER OF BLOWS REFERS TO THE NO. OF BLOWS OF A 30 LB. RAMMER DROPPED 30 INS REQ. TO DRIVE A 3" DIAM. TUBE A DIST. AS INDICATED INTO SOIL

E4

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
SOIL PROFILE AND SUMMARY OF LABORATORY TESTING

PROJECT	
SITE	
LOCATION	
HOLE	ARC 18
TECHNICIAN	
DATE	1 NOV 1960

PLOT OF ATTERBERG LIMITS & NATURAL MOISTURE CONTENTS

10 20 30 40 50 60 70 80 90%

DEPTH FT	SAMPLE	GROUND SURFACE	NUMBER OF BLOWS	MOIST. CONT. %	UNC. COMP. STR. T/SQ FT
40	U-11				
42	U-12	TRACES OF SAND			
44		SANDSTONE			
46		BOULDERS, GRAVEL, GLACIAL TILL			
48					
50					
52					
54					
56		END OF HOLE			

NOTE: SAMPLES NOT SUITABLE FOR STRENGTH TESTS

NOTE;
- ⊡ PLASTIC LIMIT
- △ LIQUID LIMIT
- ⊙ NATURAL MOISTURE CONTENT

G.W.L. INDICATES GROUND WATER LEVEL AT TIME TEST HOLE WAS DRILLED

NUMBER OF BLOWS REFERS TO THE NO OF BLOWS OF A 30 LB. RAMMER DROPPED 30 INS REQ. TO DRIVE A 3" DIAM. TUBE A DIST. AS INDICATED INTO SOIL

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

SOIL PROFILE AND SUMMARY OF LABORATORY TESTING

PROJECT	LITTLE SMOKY INVESTIGATION
SITE	
LOCATION	
HOLE	ARC 2
TECHNICIAN	RESEARCH COUNCIL OF ALTA.
DATE	26 Oct 1960

PLOT OF ATTERBERG LIMITS & NATURAL MOISTURE CONTENTS

10 20 30 40 50 60 70 80 90%

DEPTH FT	SAMPLE	GROUND SURFACE	NUMBER OF BLOWS	MOIST. CONT. %	UNC. COMP. STR. T/SQ FT.
		CLAY			
2					
4					
	4-1				
6		SAND POCKETS			
8					
10	4-2				
12		GREY BROWN			
14	4-3				
16					
18		SILT AND SAND SEAMS			
20	4-4	▽ G.W.L.			
	5				
22	6				
	7				
24	8	DARK GREY VERY DENSE			
26					
28					
30					
32	4-9	SILT AND SAND TRACES			
	10				
34	11				
	12				
36					
38					

NOTE;
- ⊡ PLASTIC LIMIT
- △ LIQUID LIMIT
- ⊙ NATURAL MOISTURE CONTENT

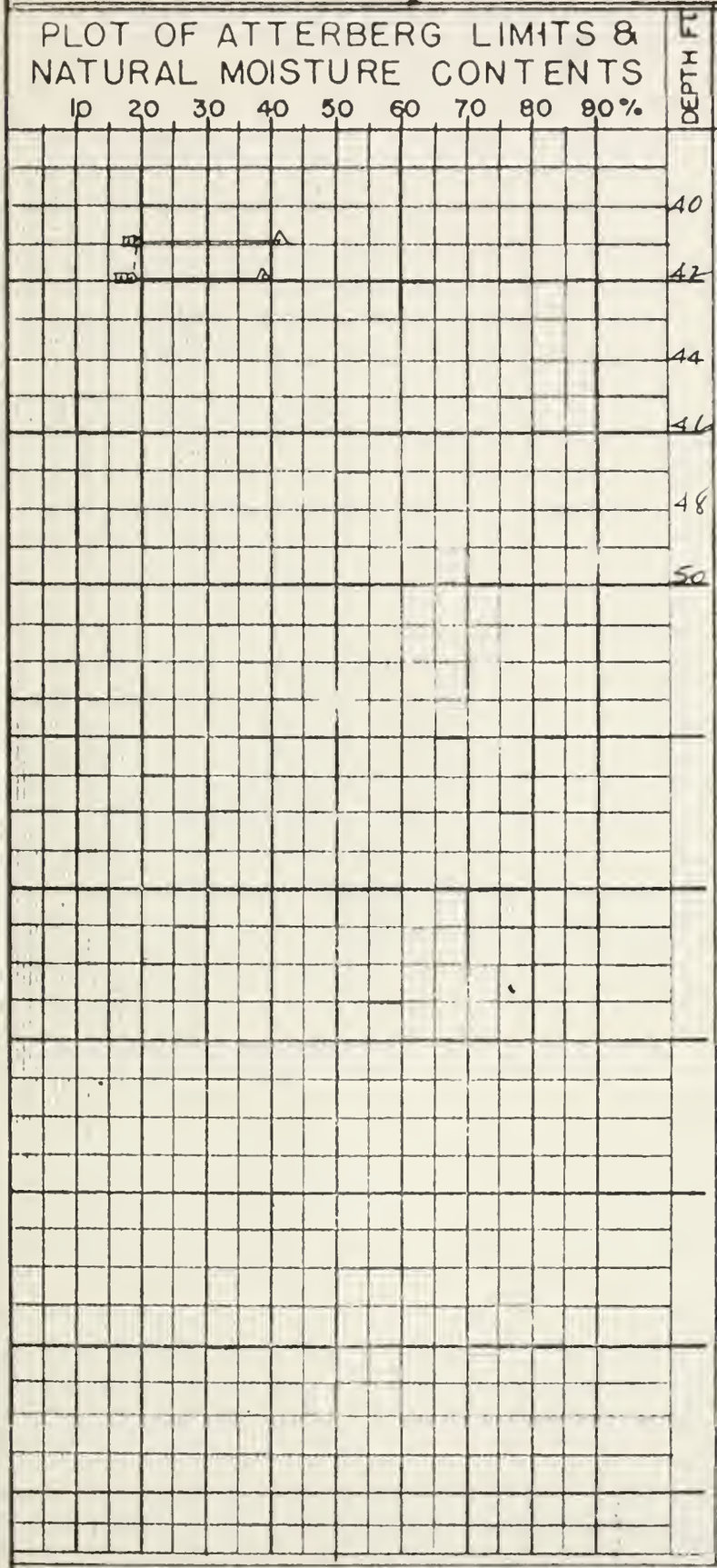
G. W. L. INDICATES GROUND WATER LEVEL AT TIME TEST HOLE WAS DRILLED

NUMBER OF BLOWS REFERS TO THE NO OF BLOWS OF A 30 LB. RAMMER DROPPED 30 INS REQ. TO DRIVE A 3" DIAM. TUBE A DIST. AS INDICATED INTO SOIL

E6

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
SOIL PROFILE AND SUMMARY OF LABORATORY TESTING

PROJECT	
SITE	
LOCATION	
HOLE	ARC 2
TECHNICIAN	
DATE	26 OCT 1960



SAMPLE	GROUND SURFACE	NUMBER OF BLOWS	MOIST. CONT. %	UNC. COMP. STR. T/SQ FT
U-13	GRAVEL			
	NOTE: SAMPLES TOO FRIABLE FOR STRENGTH TESTS			
	DENSE SILTY SHALE FROM 50 → 70 FT. – END OF HOLE.			

NOTE;
- ⊡ PLASTIC LIMIT
- △ LIQUID LIMIT
- ⊙ NATURAL MOISTURE CONTENT

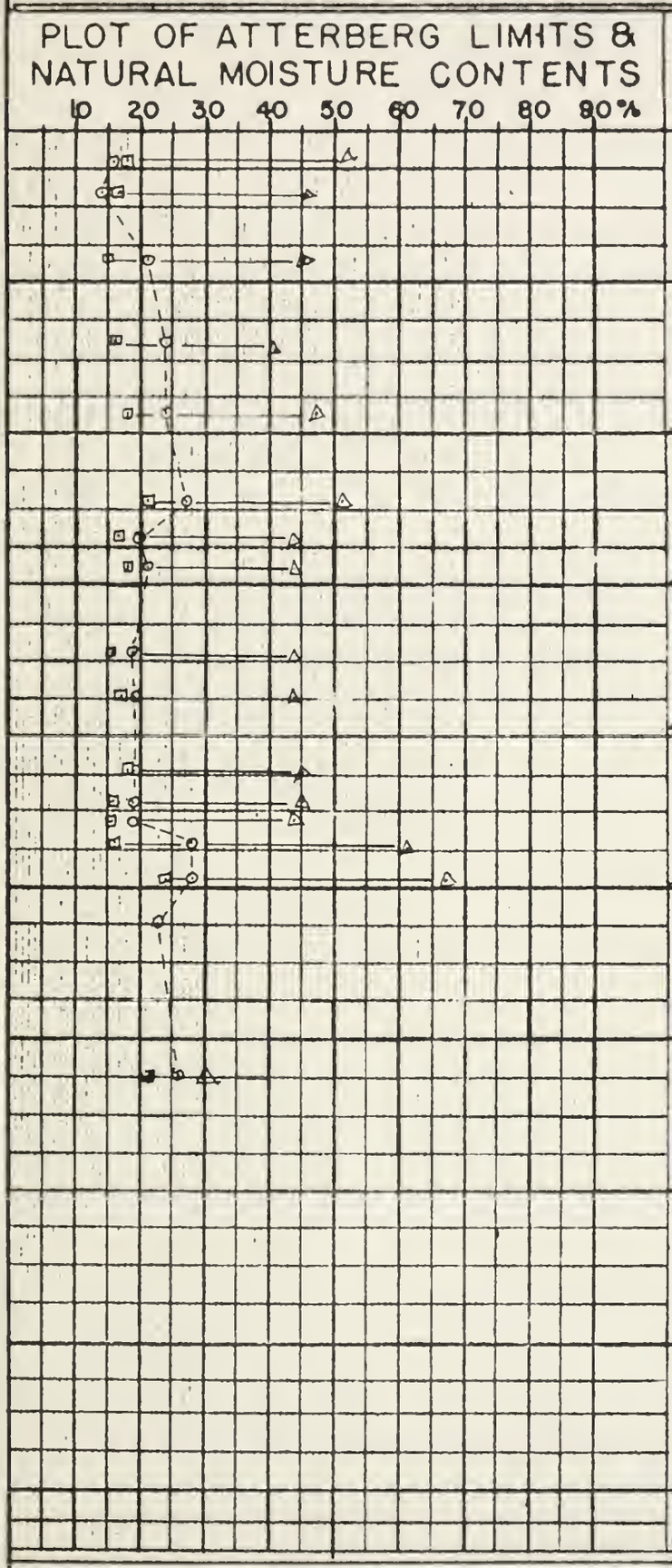
G. W. L. INDICATES GROUND WATER LEVEL AT TIME TEST HOLE WAS DRILLED

NUMBER OF BLOWS REFERS TO THE NO. OF BLOWS OF A 30 LB. RAMMER DROPPED 30 INS REQ. TO DRIVE A 3" DIAM. TUBE A DIST. AS INDICATED INTO SOIL

E 7

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
SOIL PROFILE AND SUMMARY OF LABORATORY TESTING

PROJECT LITTLE SMOKY INVESTIGATION
SITE
LOCATION
HOLE HRC 3
TECHNICIAN RESEARCH COUNCIL OF ALTA
DATE 27 OCT 1960



SAMPLE	GROUND SURFACE	NUMBER OF BLOWS	MOIST. CONT. %	UNC. COMP. STR. T/SQ FT
	CLAY		SWELLING PRESSURE T/SQ FT.	
U.1	MEDIUM PLASTIC			
	INORGANIC			
U.2	DENSE			
	DARK GREY			
U.3	MOIST			
U.4	SILT LENS			
5	PEBBLES			
6			0.65	
7			0.43	
8			0.90	
U.9			1.50	5.5
10				
11			0 85	6.5
12				
13				
U.14				
15	▽ G.W.L.			
16	SAND			
	SATURATED			
	YELLOW			
U.17	SANDSTONE			
18				
	SAND			
	DENSE			
	GREY CLAY SEAMS			
	GRAVEL			

NOTE;
- ⊡ PLASTIC LIMIT
- △ LIQUID LIMIT
- ⊙ NATURAL MOISTURE CONTENT

G. W. L INDICATES GROUND WATER LEVEL AT TIME TEST HOLE WAS DRILLED

NUMBER OF BLOWS REFERS TO THE NO OF BLOWS OF A 30 LB. RAMMER DROPPED 30 INS REQ. TO DRIVE A 3" DIAM. TUBE A DIST. AS INDICATED INTO SOIL

E8

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
SOIL PROFILE AND SUMMARY OF LABORATORY TESTING

PROJECT	
SITE	
LOCATION	
HOLE	ARC 3
TECHNICIAN	
DATE	27 OCT 1960

PLOT OF ATTERBERG LIMITS & NATURAL MOISTURE CONTENTS
10 20 30 40 50 60 70 80 90%

DEPTH FT	SAMPLE	GROUND SURFACE	NUMBER OF BLOWS	MOIST. CONT. %	UNC. COMP. STR. T/SQ FT.
100		SANDSTONE			
110					
115		END OF HOLE			

NOTE;
- ⊡ PLASTIC LIMIT
- △ LIQUID LIMIT
- ⊙ NATURAL MOISTURE CONTENT

G. W. L INDICATES GROUND WATER LEVEL AT TIME TEST HOLE WAS DRILLED

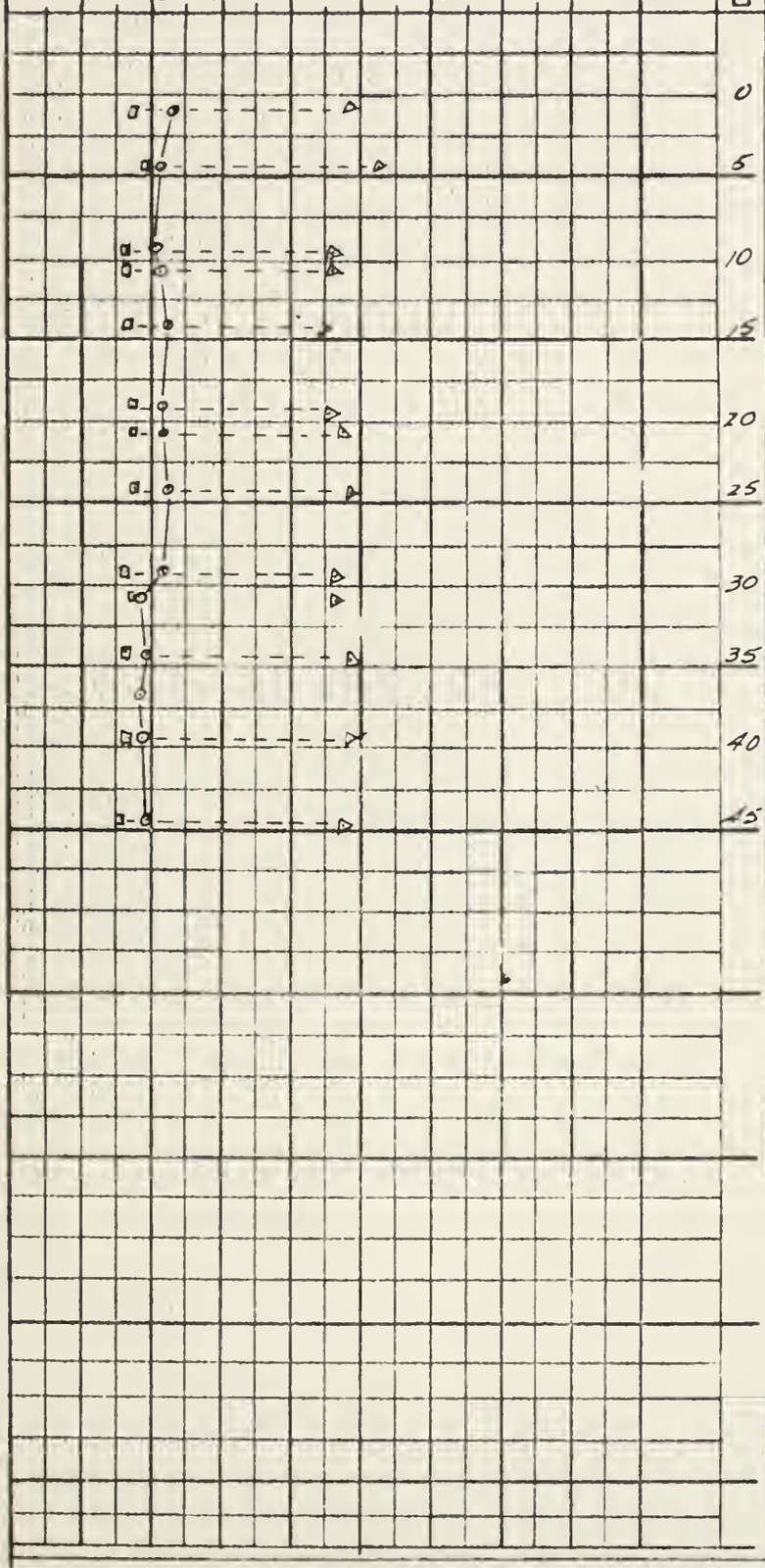
NUMBER OF BLOWS REFERS TO THE NO OF BLOWS OF A 30 LB. RAMMER DROPPED 30 INS REQ. TO DRIVE A 3" DIAM. TUBE A DIST. AS INDICATED INTO SOIL

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
SOIL PROFILE AND SUMMARY OF LABORATORY TESTING

PROJECT	THESIS
SITE	LITTLE SMOKY RIVER
LOCATION	
HOLE	ARC 3A
TECHNICIAN	RJR
DATE	7 JUN 65

PLOT OF ATTERBERG LIMITS & NATURAL MOISTURE CONTENTS

10 20 30 40 50 60 70 80 90%



DEPTH FT.	SAMPLE	GROUND SURFACE	NUMBER OF BLOWS		MOIST. CONT. %	UNC. COMP. STR. T/SQ.FT.
0		SILTY CLAY MEDIUM PLASTIC DARK GREY DENSE	12"	21		— (3.0)[3]
15		SILTY CLAY WITH PEBBLES, MEDIUM PLASTIC, DARK BROWN, DENSE	12"	30	19.8	2.4 (2.0)
25		DITTO	12"	26	20.4	2.49 (3.3)
35	S-4 S-1 S-3 S-6 S-2	DARK GREY SILTY CLAY WITH PEBBLES, DENSE, MEDIUM PLASTIC.				

NOTE:

(1) NO GROUND WATER OBSERVED

(2) DRY DRILL.

(3) — (3.3) IS POCKET PENETROMETER VALUE.

NOTE;
- ⊡ PLASTIC LIMIT
- △ LIQUID LIMIT
- ⊙ NATURAL MOISTURE CONTENT

G.W.L. INDICATES GROUND WATER LEVEL AT TIME TEST HOLE WAS DRILLED

NUMBER OF BLOWS REFERS TO THE NO. OF BLOWS OF A 30 LB. RAMMER DROPPED 30 INS REQ. TO DRIVE A 3" DIAM. TUBE A DIST. AS INDICATED INTO SOIL

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
SOIL PROFILE AND SUMMARY OF LABORATORY TESTING

PROJECT LITTLE SMOKY INVESTIGATION
SITE
LOCATION
HOLE ARC 4
TECHNICIAN RESEARCH COUNCIL OF ALTA
DATE

PLOT OF ATTERBERG LIMITS & NATURAL MOISTURE CONTENTS
10 20 30 40 50 60 70 80 90 %

DEPTH FT	SAMPLE	GROUND SURFACE	NUMBER OF BLOWS	MOIST. CONT. %	UNC. COMP. STR. T/SQ.FT.
		CLAY			
		ORGANIC SILTY			
5	4-1	BROWN GREY			
		HIGH PLASTIC			
10	4-2	PLASTIC			
15	4-3				
20	4-4				
25					
30	4-1A	DARK GREY			
	2A	TOUGH			
	3A				
35	4A				
	5A				
40	6A	SAND SEAM			
	7A				
	8A				
45	9A				
	10A				
50	11A				
	12A				
55	13A	DARK GREY CLAY			
	14A	SHALE			
60	15A				
65		SAND			
		YELLOW			
70		CLAY SEAM			
75					
80		LOSS OF CIRCULATION			
85		END OF HOLE			
		NOTE: HOLE DRY ALL THE WAY. SAMPLES UNSUITABLE FOR STRENGTH TESTS			

NOTE;
- ⊡ PLASTIC LIMIT
- △ LIQUID LIMIT
- ⊙ NATURAL MOISTURE CONTENT

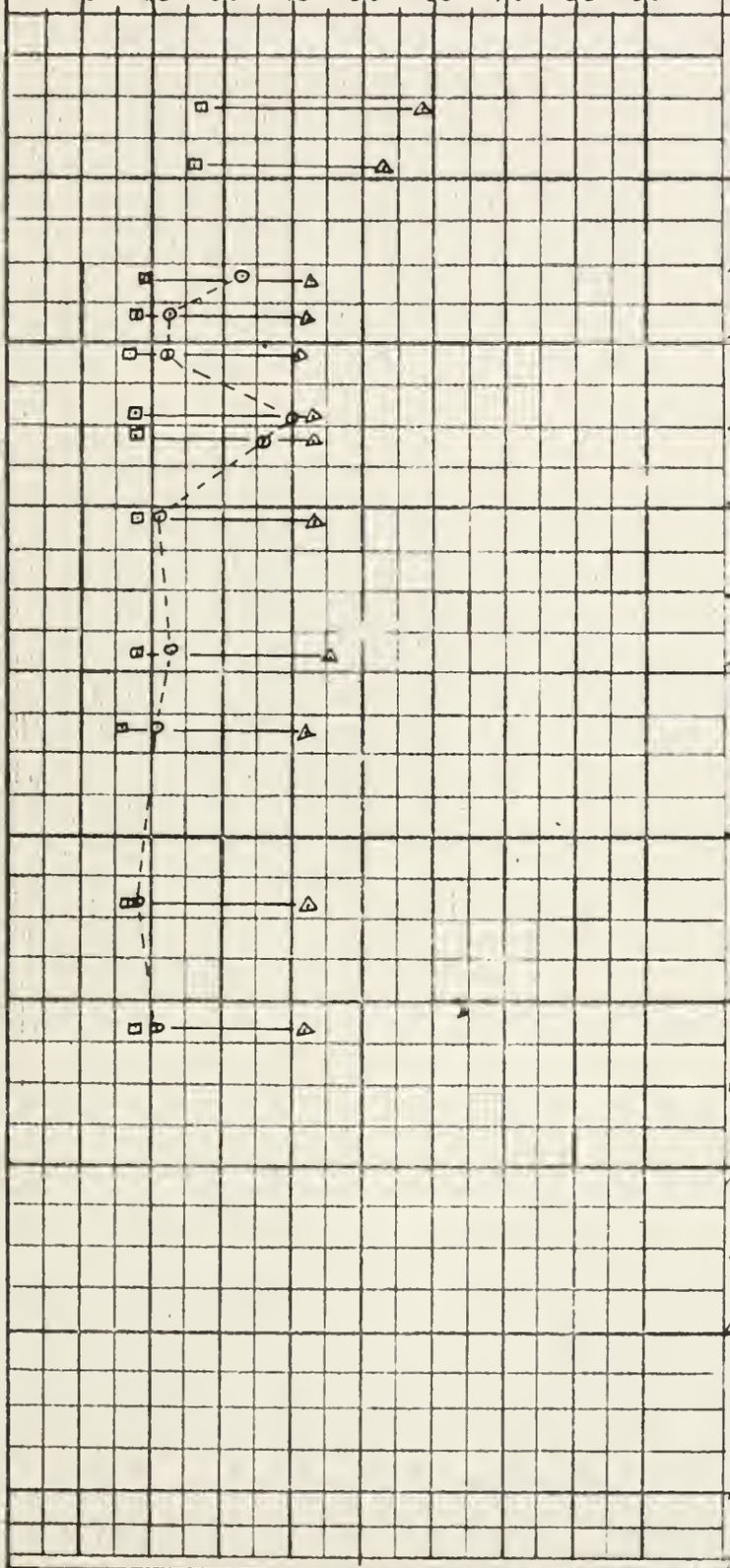
G. W. L. INDICATES GROUND WATER LEVEL AT TIME TEST HOLE WAS DRILLED

NUMBER OF BLOWS REFERS TO THE NO. OF BLOWS OF A 30 LB. RAMMER DROPPED 30 INS REQ. TO DRIVE A 3" DIAM. TUBE A DIST. AS INDICATED INTO SOIL

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
SOIL PROFILE AND SUMMARY OF LABORATORY TESTING

PROJECT LITTLE SMOKY INVESTIGATION
SITE
LOCATION
HOLE ARC 5
TECHNICIAN RESEARCH COUNCIL OF ALTA.
DATE 27 Oct 1960

PLOT OF ATTERBERG LIMITS & NATURAL MOISTURE CONTENTS
10 20 30 40 50 60 70 80 90 %



DEPTH FT.	SAMPLE	GROUND SURFACE	NUMBER OF BLOWS	MOIST. CONT. %	UNC. COMP. STR. T/SQ FT.
		CLAY			
		HIGHLY PLASTIC			
5		ORGANIC			
		DENSE			
10		G.W.L.			
15	4-1	GYPSUM CRYSTALS			
20	4-2	GREY BROWN			
	4-3				
25	4-4	WET, SOFT SOIL			
30	4-6				
	4-7				
35	4-8				
40	4-9				
	4-10				
45					
50		GRAVELLY			
	4-11				
55		SANDY			
60	4-12				
65					
70		VERY DENSE			
75		SANDSTONE			
		SOFT			
80		PEBBLES			
85		POCKETS OF BROWN GREY CLAY			
90					
95					

NOTE;
- ⊡ PLASTIC LIMIT
- △ LIQUID LIMIT
- ⊙ NATURAL MOISTURE CONTENT

G. W. L. INDICATES GROUND WATER LEVEL AT TIME TEST HOLE WAS DRILLED

NUMBER OF BLOWS REFERS TO THE NO OF BLOWS OF A 30 LB. RAMMER DROPPED 30 INS REQ. TO DRIVE A 3" DIAM. TUBE A DIST. AS INDICATED INTO SOIL

E 12

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
SOIL PROFILE AND SUMMARY OF LABORATORY TESTING

PROJECT	
SITE	
LOCATION	
HOLE	ARC 5
TECHNICIAN	
DATE	27 OCT. 1960

PLOT OF ATTERBERG LIMITS & NATURAL MOISTURE CONTENTS
10 20 30 40 50 60 70 80 90%

DEPTH FT.	SAMPLE	GROUND SURFACE	NUMBER OF BLOWS	MOIST. CONT. %	UNC. COMP. STR. T/SQ.FT.
100					
105					
110					
115		END OF HOLE			

NOTE: MOST SAMPLES NOT SUITABLE FOR STRENGTH TESTS

HOLE DRILLED IN SWAMPY AREA.

NOTE;
- ⊡ PLASTIC LIMIT
- △ LIQUID LIMIT
- ⊙ NATURAL MOISTURE CONTENT

G. W. L. INDICATES GROUND WATER LEVEL AT TIME TEST HOLE WAS DRILLED

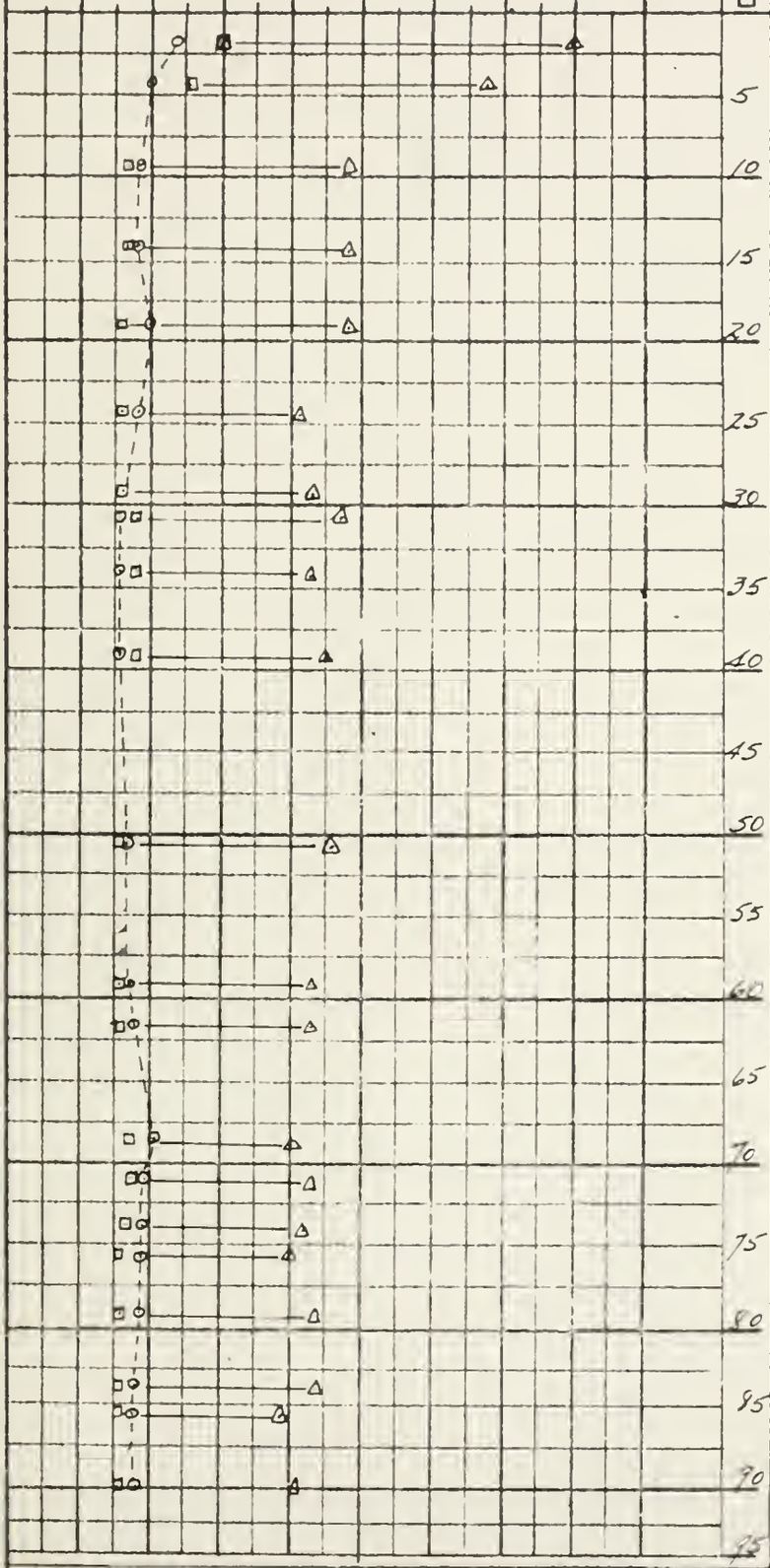
NUMBER OF BLOWS REFERS TO THE NO OF BLOWS OF A 30 LB. RAMMER DROPPED 30 INS REQ. TO DRIVE A 3" DIAM. TUBE A DIST. AS INDICATED INTO SOIL

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
SOIL PROFILE AND SUMMARY OF LABORATORY TESTING

PROJECT	LITTLE SMOKY INVESTIGATION
SITE	
LOCATION	
HOLE	ARC 6
TECHNICIAN	RESEARCH COUNCIL OF ALTA
DATE	27 OCT 1960

PLOT OF ATTERBERG LIMITS & NATURAL MOISTURE CONTENTS

10 20 30 40 50 60 70 80 90%



DEPTH FT	SAMPLE	GROUND SURFACE	NUMBER OF BLOWS	MOIST. CONT. %	UNC. COMP. STR. T/SQ.FT.
		CLAY			
		PEBBLES			
		DARK GREY			
20	U-1				
	2				
	3	SMALL PEBBLES			
30	U-4				
35	U-5				
40	U-6				
60	U-7				
	8				
	9				
		▽ SAND SEAM G.W.L			
70	U-10				
		GRAVEL			
75	U-11				
85	U-12				
95	U-13				

NOTE;
- ⊡ PLASTIC LIMIT
- △ LIQUID LIMIT
- ⊙ NATURAL MOISTURE CONTENT

G. W. L INDICATES GROUND WATER LEVEL AT TIME TEST HOLE WAS DRILLED

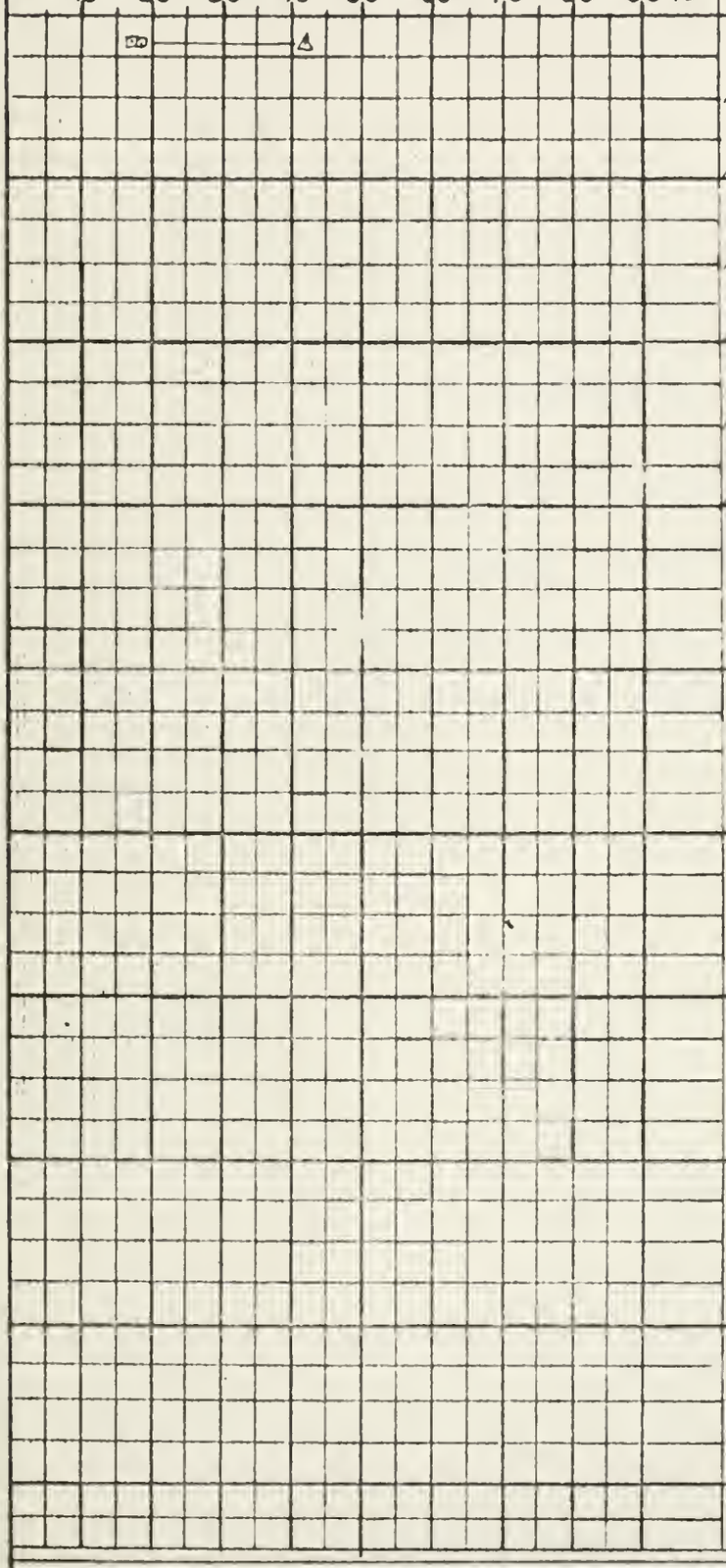
NUMBER OF BLOWS REFERS TO THE NO OF BLOWS OF A 30 LB. RAMMER DROPPED 30 INS REQ. TO DRIVE A 3" DIAM. TUBE A DIST. AS INDICATED INTO SOIL

E 17

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
SOIL PROFILE AND SUMMARY OF LABORATORY TESTING

PROJECT	
SITE	
LOCATION	
HOLE	ARC 6
TECHNICIAN	
DATE	27 OCT 1960

PLOT OF ATTERBERG LIMITS & NATURAL MOISTURE CONTENTS
10 20 30 40 50 60 70 80 90 %



DEPTH FT	SAMPLE	GROUND SURFACE	NUMBER OF BLOWS	MOIST. CONT. %	UNC. COMP. STR. T/SQ FT
		HOLE ADVANCED FROM THIS DEPTH			
100					
105					
110					
115					
120					
125		▽ G.W.L			
		SAND			
		CLAY, SILTY			
		SANDSTONE AT 355'			
		END HOLE AT 360'			
		NOTE: MOST SAMPLES NOT SUITABLE FOR STRENGTH TESTS			

NOTE;
- ⊡ PLASTIC LIMIT
- △ LIQUID LIMIT
- ⊙ NATURAL MOISTURE CONTENT

G. W. L. INDICATES GROUND WATER LEVEL AT TIME TEST HOLE WAS DRILLED

NUMBER OF BLOWS REFERS TO THE NO OF BLOWS OF A 30 LB. RAMMER DROPPED 30 INS REQ. TO DRIVE A 3" DIAM. TUBE A DIST. AS INDICATED INTO SOIL

Appendix F

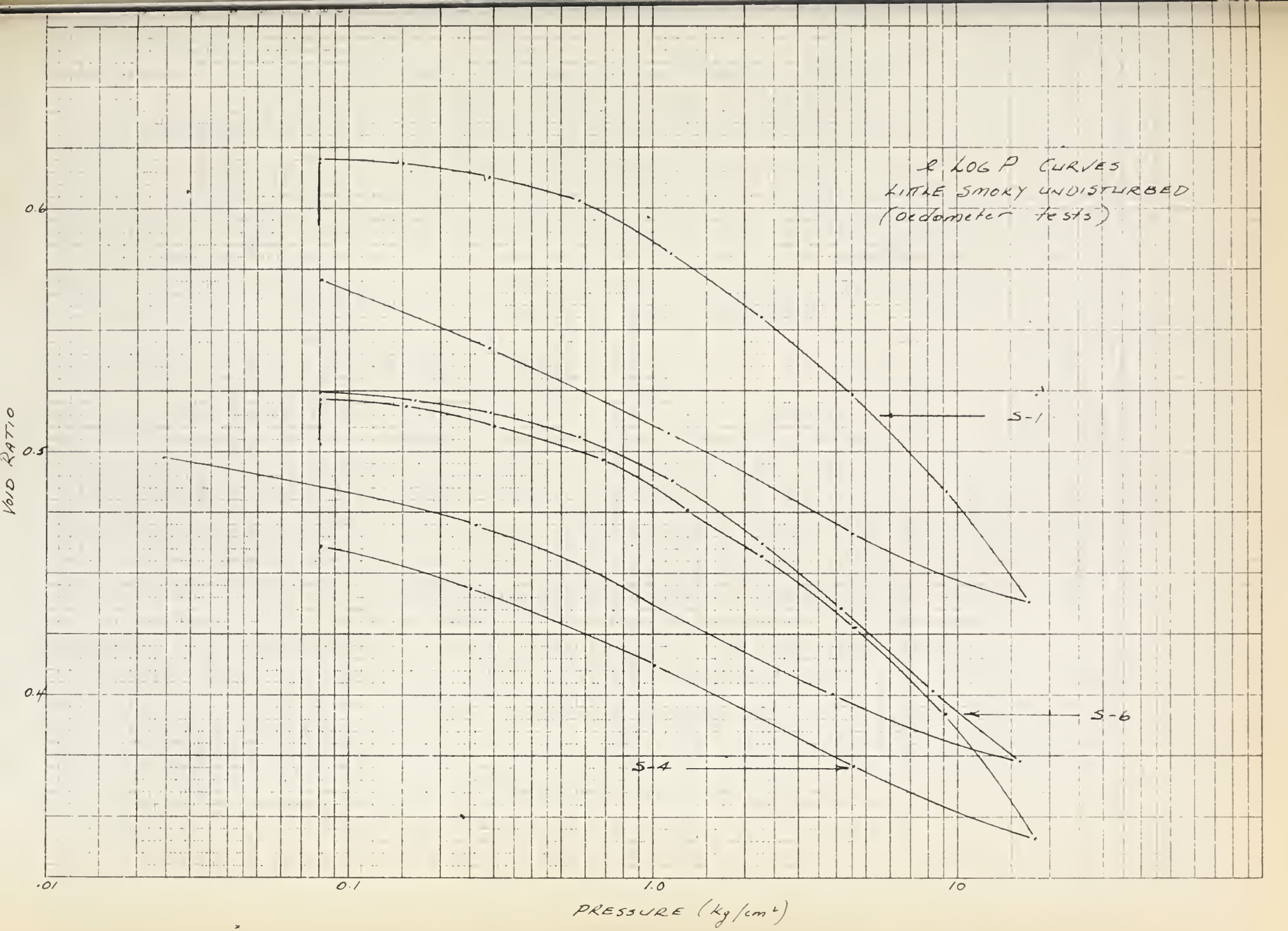
TYPICAL e log P CURVES

SAMPLE DATA SHEETS FOR

a. DIRECT SHEAR TEST

b. TRIAXIAL COMPRESSION UNDISTURBED

c. TRIAXIAL COMPRESSION REMOULDED


e LOG P CURVES
LITTLE SMOKY UNDISTURBED
(oedometer tests)
S-1
S-6
S-4
0.6
0.5
0.4
VOID RATIO
.01
0.1
1.0
10
PRESSURE (kg/cm²)

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING

SOIL MECHANICS LABORATORY

DIRECT SHEAR TEST

STRAIN RATE: 0.1-INCH PER DAY (24hrs)

PROJECT	THESIS
SITE	LITTLE SMOKY RIVER
SAMPLE	S1 TRIAL NO.
LOCATION	LITTLE SMOKY RIVER.
HOLE	ARC 3A DEPTH
TECHNICIAN	RJR DATE 4 Aug '65.

SHEAR BOX:		INITIAL GROSS WEIGHT	—	SAMPLE DESCRIPTION	
WIDTH	2.0 in.	FINAL GROSS WEIGHT	—	DARK GREY, STIFF CLAY TILL	
LENGTH	3.88 in.	WEIGHT OF SPECIMEN	821.1 gm.	w_L = 48%, w_P = 17%, w_S = 10%	
INITIAL AREA	7.76 in²	VOID RATIO, e	0.5	43% < 24.	
HEIGHT OF SPECIMEN	3.0 in.	TOTAL NORMAL LOAD	161.39 lbs.	G_s 2.71	D_{10} — n —
VOLUME OF SPECIMEN	23.28 in³	UNIT NORMAL LOAD σ	1.5 Tsf.	Cu —	

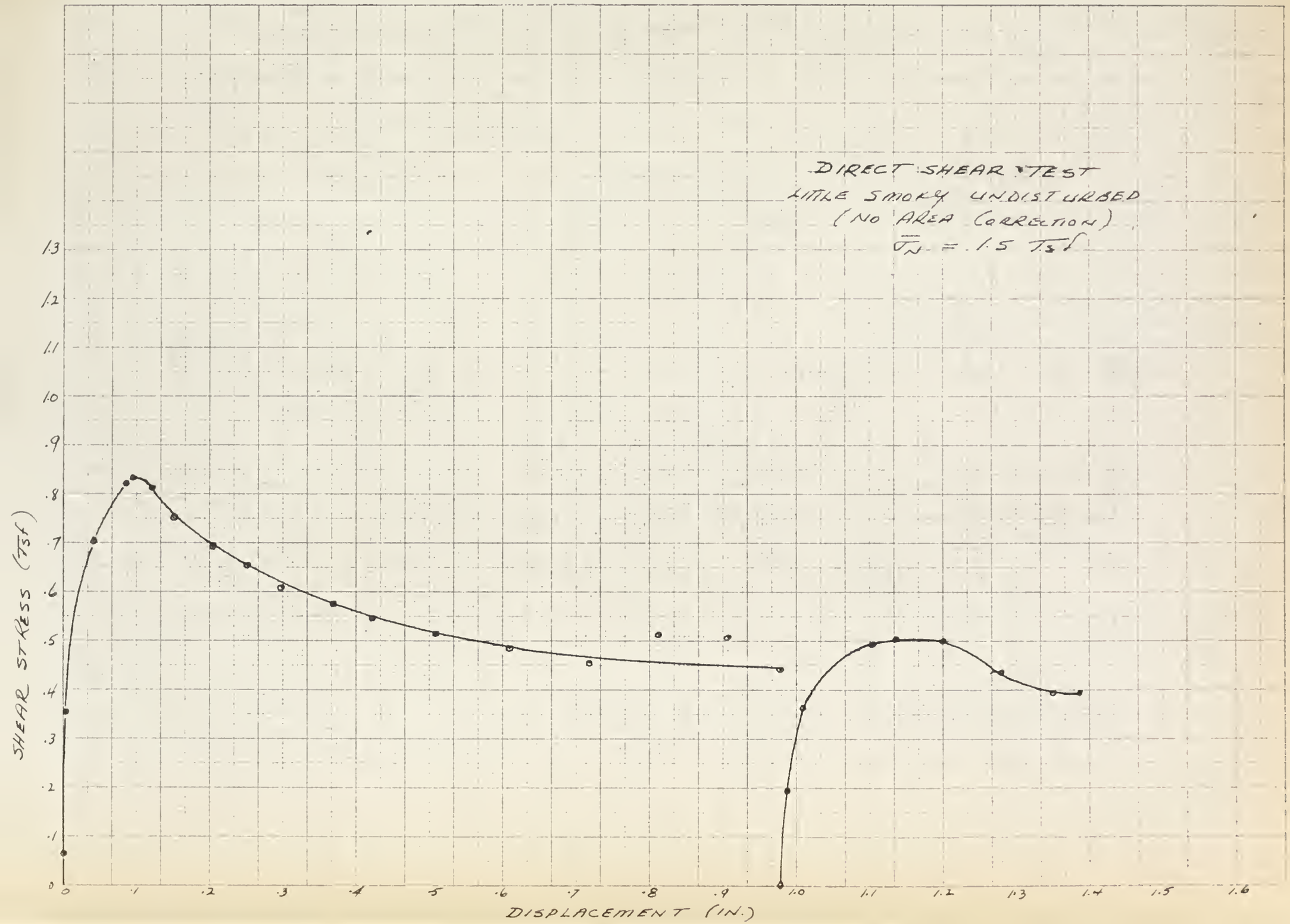
Horizontal ~~Dial~~ Movement: Time	Horizontal ~~Dial~~ Movement: Inches	Load ~~Proving~~ Cell ~~Ring~~ ~~Dial~~ Reading millivolts	Vertical Dial Reading in × 10⁻⁴	Vertical ~~Strain~~ Defn. in × 10⁻⁴	Total Horizontal Shear ~~Def.~~ Volts	~~Def.~~ in	Sample Length in	Cross-Section Area in²	Total Shear Force * lbs.	NO AREA CORRECTION Shear Stress τ Tsf.	FULL AREA CORRECTION τ / ~~σ~~ Tsf
~~Time~~ 04 1400		0	614	0	3.916	0	3.88	15.52	0	0	0
05 0100		- 3.66			3.758	.0376	3.84	15.36	152.3	.701	.708
0839		4.23	509.9	104.1	3.622	.07		15.24	176	.811	.826
1300		4.32			3.517	.095	3.79	15.16	179.7	.828	.848
06 0300		3.85			3.271	.154		14.90	160.2	.736	.768
1313		3.61	454.1	159.9	3.059	.204		14.70	150.2	.691	.730
07 1100		3.16	427.1	186.9	2.659	.299	3.58	14.32	131.5	.604	.655
08 1200		2.91			2.194	.410		13.88	121.1	.556	.622
09 1000		2.66			1.781	.508	3.37	13.48	110.7	.508	.585
10 0800		2.50	415.6	198.4	1.359	.609	3.27	13.08	104	.477	.566
11 0400		2.25			.983	.698	3.18	12.72	93.6	.429	.523
13 1829		2.29	364.1	247.1	-.204	.981	2.90	11.60	95.3	.437	.584
				REVERSE DIRECTION							
1852		+.04	746.8	247.1	+.361	.981	2.90	11.60	0	0	0
14 1500		- 2.52	670.4	325.5	3.308	1.053	2.971	11.88	103.2	.473	.618
15 0900		2.61			2.982	1.131	3.05	12.2	106.9	.490	.624
1300		2.03			1.984	1.369	3.2 86	13.14	82.8	.379	.447

* LOAD CORRECTED FOR PISTON FRICTION = 1.2 lbs.

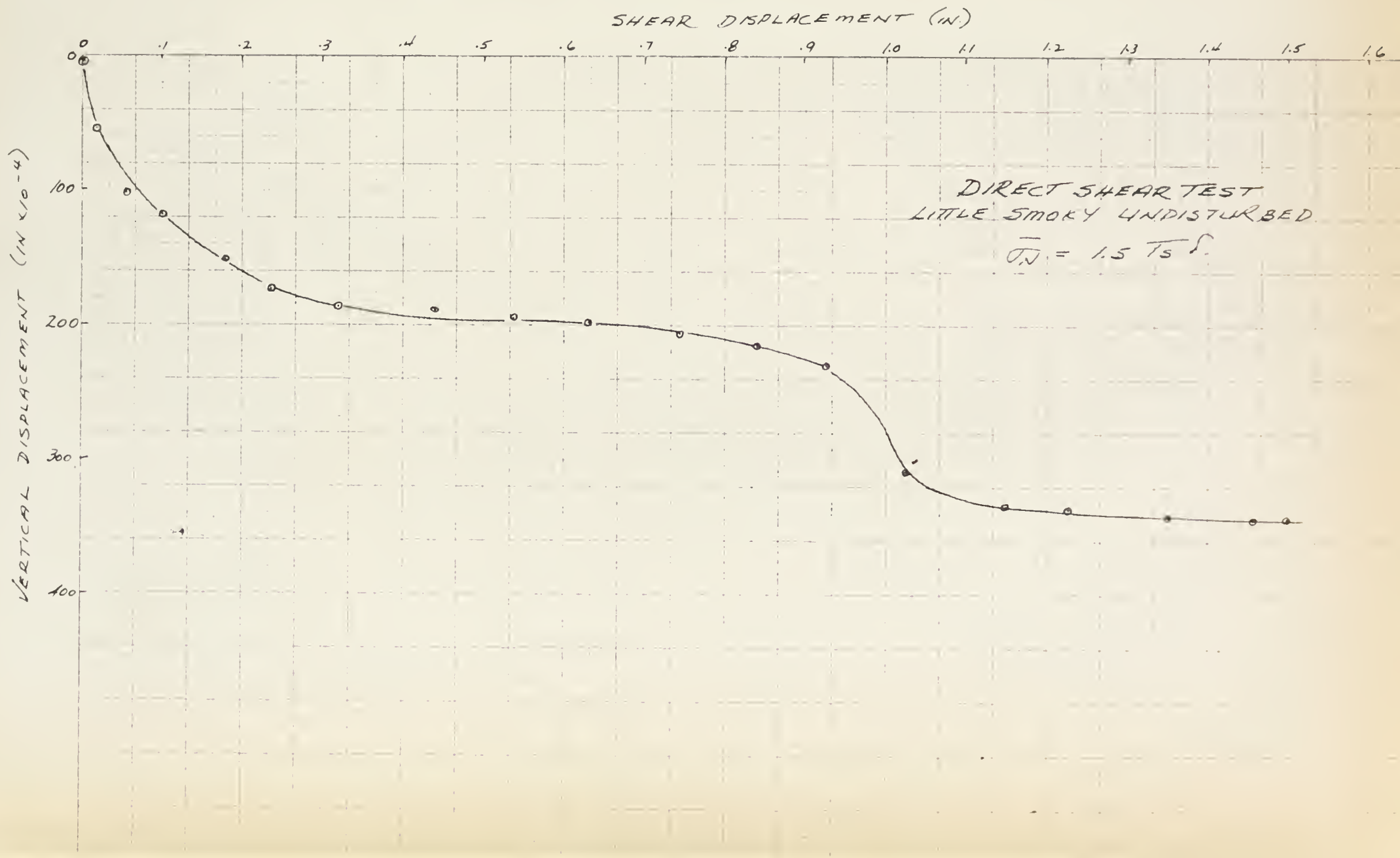
SAMPLE TRIAL SHEET OF

SHEET OF

KEUFFEL & ESSER CO



K&E 10 X 10 TO THE CENTIMETER 46 1513
KEUFFEL & ESSER CO.



CONSOLIDATION
DIRECT SHEAR BOX
LITTLE SMOKY UNDISTURBED
ARC 3A S-1
σ_N' = 1.5 Tsf.

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Sample Description STIFF, GREY-BLUE CLAY WITH PEBBLES

W_L = 48% W_P = 17% W_S = 10%

43% < 2μ.

UNDISTURBED

Initial Weight 194.07 gms

Project THESIS

Hole No. ARC 3A

Depth 40 FT. sample 5-6

Engineer RJR

Technician

Date of Set-up 21 Oct 65

Test Lateral Pressure 1.18 kg/cm²

Length, mm	1. 80.86	2. 80.76	Aver: 80.81	Area Top	A_T 1126	sq.mm
Diam. mm Top	1. 38.0	2. 37.74	Aver 37.87	Area Centre	A_C 1128	sq.mm
Centre	1. 37.98	2. 37.82	Aver: 37.90	Area Bottom	A_B 1138	sq.mm
Bottom	1. 38.06	2. 38.08	Aver: 38.07	Average X-Sect Area		

Original Volume 91.32 cc

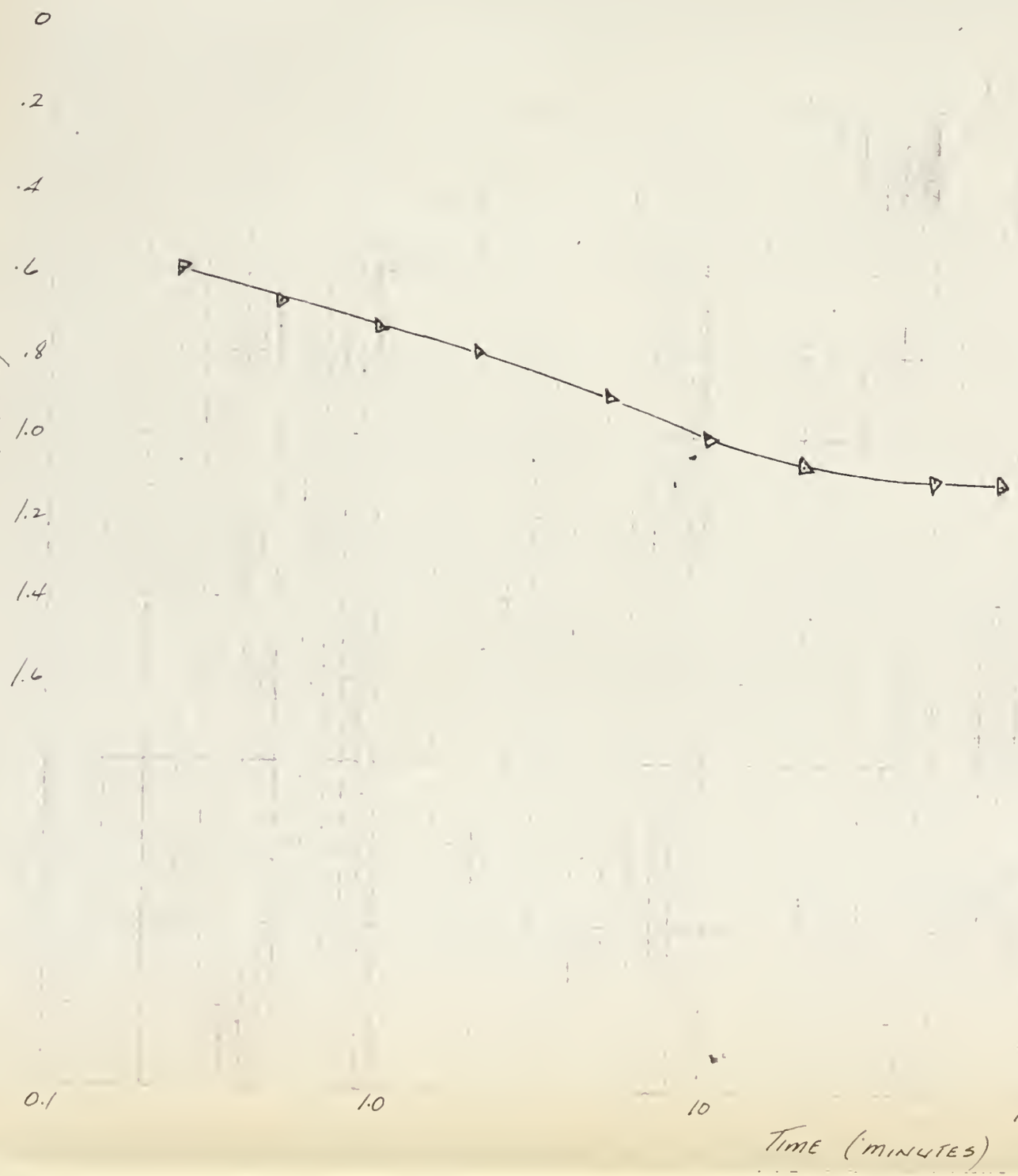
Average X-Sect Area = ¼ (A_T + 2 A_C + A_B) = 1130 mm²

CONSOLIDATION DATA

Date Time	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
1145	0	21.70	0.
	15	21.1	0.6
	30	21.02	0.68
	1	20.97	0.73
	2	20.90	0.8
	5	20.80	0.9
	10	20.70	1.00
	20	20.64	1.06
	50	20.60	1.10
LARGE AIR BUBBLES	55	18.91	1.10
	80	18.90	1.11

CONSOLIDATION DATA – cont'd

Date Time	Δ t min.	Burette Rdg. c.c.	Δ V c.c.



TRIAXIAL CONSOLIDATION
LITTLE SMOKY UNDISTURBED
ARC 3A 5-6
σ_3 = 1.18 kg/cm²

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. ARC 3A LITTLE SMOKY

Depth 40' Sample 5-6

Engineer RJR

Technician —

Date of Test 23 Oct '65

Test Lateral Pressure σ_3 1.18 kg/cm²

Back Pressure 2.18 kg/cm²

Remarks STRAIN RATE = 0.11 % PER HOUR
PROVING RING 3-1498
UNDISTURBED.

Area Correction Factor 1.008

P.P. TRANSDUCER

Data at Failure

$(\sigma_1 - \sigma_3)$ = 1.669 kg/cm² σ_1 = 2.849

P = −0.13

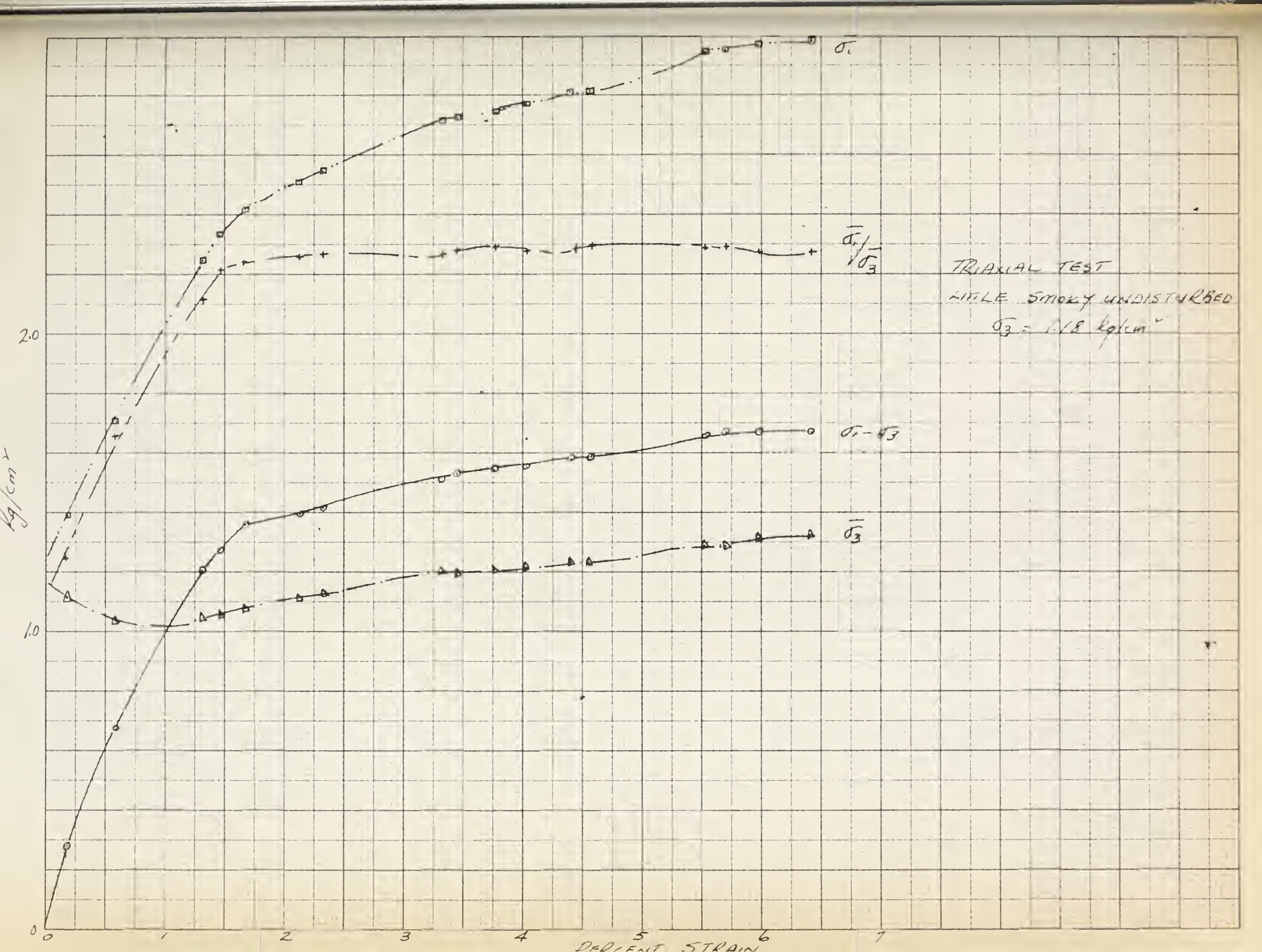
$\bar{\sigma}_1$ = 2.979

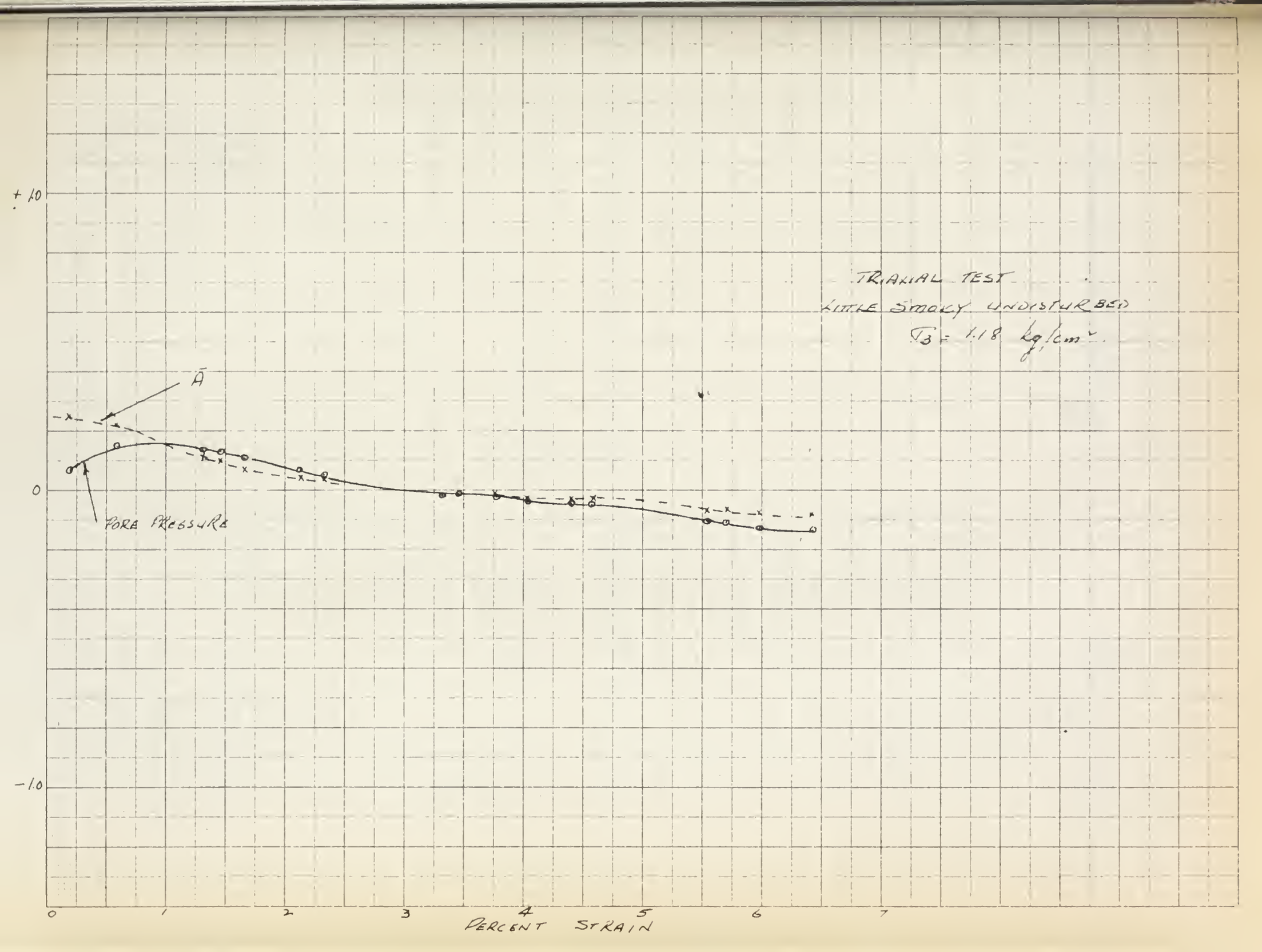
$\bar{\sigma}_3$ = 1.31

Strain 5.98 %

L_o = 80.81 cm.

Time min	Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const & kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² P_P	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}}{P_P}{\sigma_1 - \sigma_3}$
0955	0	11.30	2200		0					0	
1410	14.6	11.32	2266.8	.0465	0.271	+.069	1.388	1.111	1.249	0.18	*0.249
2055	46.8	11.37	2366.6	.0455	0.672	0.148	1.704	1.032	1.651	0.58	0.220
0830	105.8	11.45	2511.6	.0438	1.202	0.135	2.247	1.045	2.15	1.31	0.112
1035	118.1	11.47	2532.9	.0437	1.278	0.128	2.330	1.052	2.215	1.46	0.1
1315	134.5	11.49	2550	.0435	1.336	0.103	2.413	1.077	2.240	1.66	0.077
1840	171.9	11.55	2568.6	.0434	1.396	0.066	2.510	1.114	2.253	2.13	0.047
2100	188	11.57	2576.1	.0433	1.419	0.056	2.543	1.124	2.262	2.33	0.039
0815	267.9	11.69	2608.4	.0431	1.518	−.02	2.718	1.2	2.265	3.31	−.013
0950	279.1	11.70	2612.2	.0431	1.531	−.017	2.728	1.197	2.279	3.45	−.011
1307	304.2	11.74	2618	.0430	1.543	−.021	2.744	1.201	2.285	3.76	−.014
1600	325.5	11.77	2622.1	.0430	1.554	−.038	2.772	1.218	2.276	4.03	−.024
2004	355.4	11.82	2630.8	.0430	1.580	−.047	2.807	1.227	2.288	4.40	−.03
2200	369.1	11.84	2634.1	.0429	1.585	−.046	2.811	1.226	2.293	4.57	−.029
0845	447.1	11.96	2659.8	.0428	1.659	−.107	2.949	1.287	2.289	5.53	−.064
1035	460.5	11.98	2662.5	.0428	1.666	−.108	2.954	1.288	2.293	5.70	−.065
335	483.5	12.02	2666.1	.0427	1.669*	−.130	2.979	1.31	2.274	5.98	−.078
1820	518.4	12.08	2672.1	.0427	1.669	−.14	2.989	1.32	2.264	6.42	−.084


$\bar{\sigma}_1$
$\bar{\sigma}_1/\bar{\sigma}_3$
$\sigma_1 - \bar{\sigma}_3$
$\bar{\sigma}_3$
TRIAXIAL TEST
LITTLE SMOKY UNDISTURBED
$\bar{\sigma}_3$ = 1.18 kg/cm²
2.0
1.0
0
Kg/cm²
0
1
2
3
4
5
6
7
PERCENT STRAIN


+1.0
0
-1.0
TRIAXIAL TEST
LITTLE SMOKY UNDISTURBED
σ3 = 1.18 kg/cm²
A
PORE PRESSURE
0
1
2
3
4
5
6
7
PERCENT STRAIN

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING

SOIL MECHANICS LABORATORY
TRIAXIAL COMPRESSION TEST COMPUTATIONS

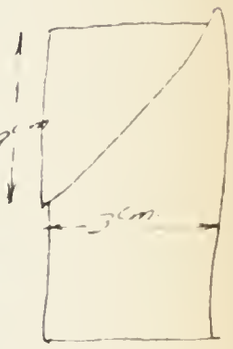
Sample No. 5-6
Sample Desc. STIFF, GREY CLAY W/ PEBBLES

BEGINNING OF TEST

Original Vol of Specimen 91.32 c.c
Wt. of Soil Solids in Specimen 164.19 gm.
Vol. of Soil Solids 60.59 c.c
Vol of Voids 30.73 c.c
Original Void Ratio, e_o 0.507
Original Porosity, n, 33.64 %
Wt. of Water 29.83 gms
Original Degree of Saturation, S, 97.1 %
Original Wet Wt., lb/cu. ft. 132.6
Original Dry Wt., lb/cu. ft. 112.2

END OF TEST

Final Vol of Specimen (by Hg immersion) 92 c.c
Vol. of Soil Solids 60.57 c.c
Vol. of Voids 31.43 c.c
Final Void Ratio, e_f 0.519
Final Porosity, n, 34.2 %
Wt. of Water 31.26 gms
Final Degree of Saturation 99.5 %
Final Wet Wt. lb/cu.ft. 132.5
Final Dry Wt. lb/cu.ft. 111.3



Wet wt. of specimen at beginning of test gm. 194.02
Wet wt. of specimen at end of test gm. 195.41
Weight loss Skin gm. 1.39
Vol. change from burette rdg. discrepancy -.11 cc. 2.50 (1394 III)

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Sample Description GREY, CLAYEY TILL
$w_L = 48\%$, $w_p = 17\%$,
$w_s = 10\%$, $13\% < 2\mu$.
CONSOLIDATED FROM $w_0 > w_L$ TO $\approx$
1 kg/cm².

Initial Weight 139.90 gms

Project THESIS

Hole No. ARC 3A ~~LITTLE SMOKY~~

Depth ~~sample~~ REMOULDED

Engineer R.J.R.

Technician —

Date of Set-up 15 Oct '65

Test Lateral Pressure 5.2 kg/cm²

Length, mm	1. 75.30	2. 75.28	Aver: 75.29	Area Top	A_T 1057	sq.mm
Diam. mm Top	1. 36.82	2. 36.58	Aver: 36.70	Area Centre	A_C 1039	sq.mm
Centre	1. 36.48	2. 36.28	Aver: 36.38	Area Bottom	A_B 1033	sq.mm
Bottom	1. 36.30	2. 36.26	Aver: 36.28	Average X-Sect Area		

Original Volume 78.45 cc

$= \frac{1}{4}(A_T + 2A_C + A_B)$

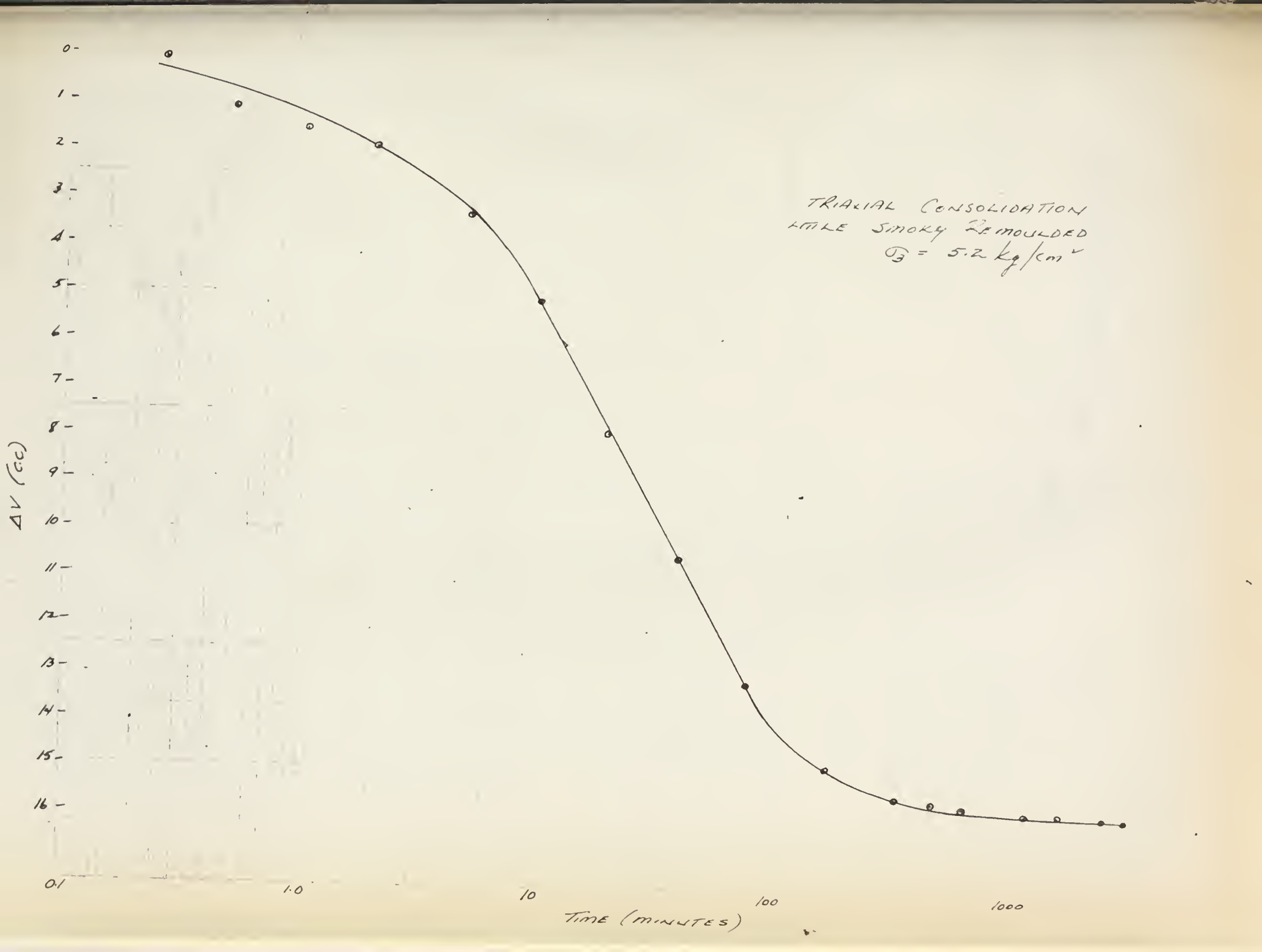
$= 1042$ mm²

CONSOLIDATION DATA

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
5 1017	0	21.11	0
	0.25	21.00	0.11
	0.5	19.94	1.17
	1.0	19.59	1.52
	2.0	19.19	1.92
	5.0	17.78	3.33
	10.0	~~13.31~~ 15.98	5.13
	20	13.22	7.89
	40	10.6	10.51
	80	7.9	13.21
1309	172	6.2	14.91
1600	343	5.6	15.51
1830	493	5.45	15.66
2135	678	5.4	15.71
0830	1233	5.31	15.80
1626	1709	5.28	15.83
0817	2660	5.24	15.87
1810	3253	5.23	15.88

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.


TRIAXIAL CONSOLIDATION
LITTLE SMOKY REMOULDED
σ_3 = 5.2 kg/cm²
ΔV (c.c)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
0.1
1.0
10
100
1000
TIME (MINUTES)

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

RIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. ARC 3A

Depth _____ Sample LITTLE SMOKY REMOULDED

Engineer RJR

Technician _____

Date of Test 18 OCT. 65

Test Lateral Pressure σ_3 5.2 kg/cm²

Back Pressure NIL

Remarks STRAIN RATE - 0.5 %/HR.
PROVING RING 3-1584

Area Correction Factor 1.164.

D.P. TRANSDUCER

ata at Failure

$(\sigma_1 - \sigma_3)$ = 3.460 kg/cm² σ_1 = 8.66

= 3.276

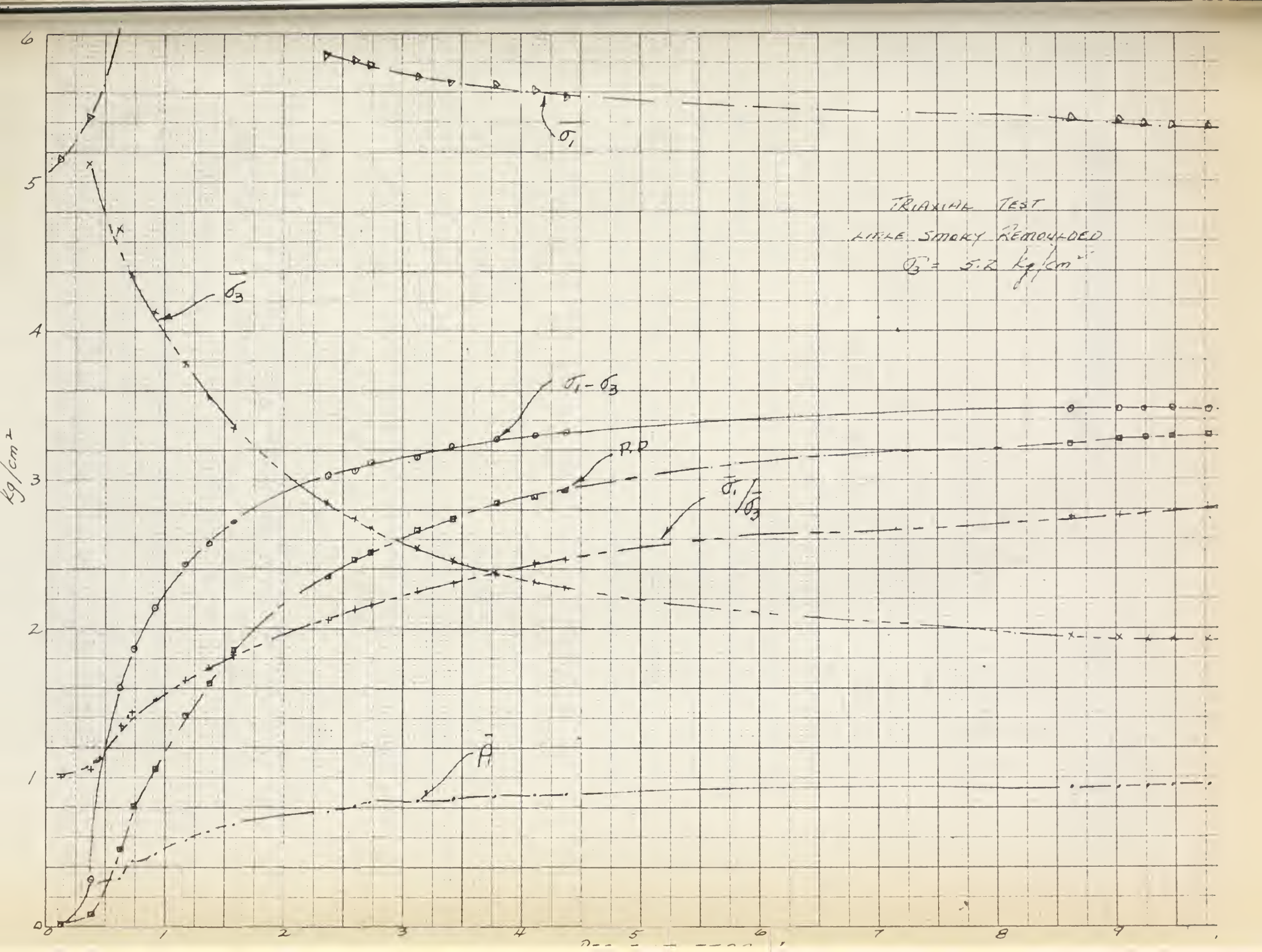
$\bar{\sigma}_1$ = 5.384

$\bar{\sigma}_3$ = 1.924

train 9.49 %

L_0 = 75.29 mm.

Time min	Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const & kg/Div	$\sigma_1 - \sigma_3 = \frac{kp \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² P_P	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}}{\sigma_1 - \sigma_3}$ P_P
1153	209.7	10.43	5.6	.101	.0063 16/141	.0371	5.169	5.163	1.001	.129	+5.889
1230	228.8	10.46	27	.1002	.3149 16/200	.0906	5.424	5.109	1.062	.382	0.288
1337	245.8	10.48	139.6	.096	1.6 16/64	.513	6.287	4.687	1.341	.608	0.321
1410	256.3	10.5	182.3	.0926	1.871 16/79	.8016	6.269	4.398	1.425	.748	0.428
1440	268.5	10.52	212.8	.0908	2.138 17/260	1.066	6.272	4.134	1.517	0.91	0.499
1522	288	10.54	245.8	.0894	2.427 17/631	1.409	6.218	3.791	1.64	1.169	.581
1553	302.4	10.56	263.9	.0888	2.58 17/874	1.639	6.141	3.561	1.725	1.36	.635
1626	319.9	10.59	280.9	.0883	2.726 18/100	1.856	6.070	3.344	1.815	1.592	.681
1815	378.9	10.67	316.6	.0873	3.015 18/441	2.368	5.847	2.832	2.065	2.376	.785
1845	395.8	10.70	323.2	.0872	3.066 18/742	2.462	5.804	2.738	2.12	2.6	.803
1907	406.9	10.71	327.6	.0871	3.101 18/801	2.52	5.781	2.68	2.157	2.748	.813
2000	435.6	10.76	336.9	.0869	3.167 18/934	2.658	5.709	2.542	2.246	3.129	.839
2035	457.1	10.79	343.1	.0868	3.213 19/019	2.741	5.672	2.459	2.307	3.415	.853
2125	485.8	10.83	349.8	.0867	3.260 19/108	2.828	5.632	2.372	2.374	3.796	.867
2205	509.5	10.87	355	.0866	3.292 19/171	2.884	5.608	2.316	2.421	4.111	.876
2240	529.3	10.90	358.5	.0865	3.312 19/210	2.933	5.579	2.267	2.461	4.373	.886
2340	848.9	11.40	393.6	.0861	3.460 19/542	3.242	5.418	1.958	2.767	8.619	.937
2430	878.1	11.45	395.2	.0860	3.455 19/552	3.252	5.403	1.948	2.774	9.006	.941
2500	895.9	11.48	396.1	.0860	3.454 19/560	3.263	5.391	1.937	2.783	9.242	.945
2530	914.8	11.51	397.8	.0860	3.460 19/574	3.276	5.384	1.924	2.798	9.493	.947
2608	936.7	11.55	398.7	.0860	3.456 19/580	3.285	5.371	1.915	2.805	9.784	.951
2645	957.2	11.59	400.1	.0860	3.456 19/589	3.288	5.368	1.913	2.807	10.056	.951
2752	998.6	11.66	401.6	.0860	3.448 19/600	3.301	5.347	1.899	2.816	10.606	.957
2824	1017.1	11.69	402.6	.0860	3.448 19/601	3.301	5.347	1.899	2.816	10.852	.957
2900	1038.1	11.72	403.4	.086	3.446 19/609	3.309	5.337	1.891	2.822	11.13	.960
2945	1067.1	11.78	405	.0859	3.438 19/611	3.311	5.327	1.889	2.820	11.52	.963
3000	1109	11.85	406.6	.0859	3.431 19/618	3.321	5.31	1.879	2.826	12.07	.968
3045	1138.1	11.90	407.8	.0859	3.426 19/619	3.315	5.311	1.885	2.818	12.46	.968
3115	1156	11.94	408.9	.0859	3.424 19/619	3.315	5.309	1.885	2.816	12.70	.968


TRIAXIAL TEST
LITTLE SMOKY REMOULDED
σ_3 = 5.2 kg/cm²
$\bar{\sigma}_1$
$\bar{\sigma}_3$
$\bar{\sigma}_1 - \bar{\sigma}_3$
P.P
$\bar{\sigma}_1 / \bar{\sigma}_3$
$\bar{p}$
Kg/cm²
0
1
2
3
4
5
6
7
8
9

UNIVERSITY OF ALBERTA
DEPARTMENT OF CIVIL ENGINEERING

SOIL MECHANICS LABORATORY
TRIAXIAL COMPRESSION TEST COMPUTATIONS

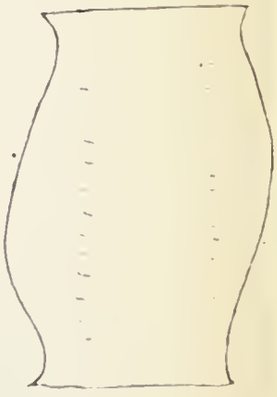
Sample No. REMOULDED
Sample Desc. GREY CLAY WITH PEBBLES

BEGINNING OF TEST

Original Vol of Specimen 35.75 c.c
Wt. of Soil Solids in Specimen 100.09 gms
Vol. of Soil Solids 36.93 c.c
Vol of Voids 41.52 c.c
Original Void Ratio, e_o 1.12
Original Porosity, n, 52.78 %
Wt. of Water 39.81 gm
Original Degree of Saturation, S, 95.88 %
Original Wet Wt., lb/cu. ft. 111.37
Original Dry Wt., lb/cu. ft. 79.77

END OF TEST

Final Vol of Specimen (by Hg immersion) 63.27 c.c
Vol. of Soil Solids 36.50 c.c
Vol. of Voids 26.77 c.c
Final Void Ratio, e_f 0.733
Final Porosity, n, 42.3 %
Wt. of Water 26.38 gm
Final Degree of Saturation 98.54 %
Final Wet Wt. lb/cu.ft. 123.59
Final Dry Wt. lb/cu.ft. 97.58



Bulging failure

Wet wt. of specimen at beginning of test 37.90 gm.
Wet wt. of specimen at end of test 25.30 gm.
Weight loss 12.60 gm.
Vol. change from burette rdg. discrepancy .588 cc. .128

B29847